A Detailed Study of Banks Peninsula Loess Shear Strength

A thesis submitted in fulfillment of the requirements
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by

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"Good view, shame about the neighbours!"

ABSTRACT

This thesis project has examined the moisture dependency of shear strength in the loess soils of Banks Peninsula. These dominant silt materials are generally regarded as having an angle of internal friction between 25 and 30°, and cohesion of zero, when the soil is saturated. However, soil behaviour in terms of slope stability would appear to support a cohesion value higher than zero as vertical cliffs of loess can be seen to stand indefinitely. It is agreed that loess soils upon saturation do show very low shear strengths, but these soils rarely become saturated due to their low permeability ($k<10^{-6}$).

To address the quantity of water content dependency of shear strength, five field sites have been sampled for unconsolidated undrained triaxial shear strength to determine c and ϕ at varying water contents in the total stress state, which most closely simulates observed behaviour. The field sites were: 1) Moncks Spur primary airfall in situ loess; 2) Stonehaven Subdivision loess-colluvium; 3) Worsleys Spur primary airfall in situ loess; 4) Duvauchelle primary airfall in situ loess; 5) Whaka Terrace uncompacted loess fill. Nominated water contents for the shear testing program were 6%, 10%, 14% and "as wet as possible". Samples were prepared in the laboratory by immersing the stainless steel sampling tube containing the soil sample in water for up to 2 weeks to a water content maximum, and then drying them back to the nominated water content.

Analysis of results of all five field sites tested at the four nominated water contents showed a new cohesion trend, which has not previously been reported in Banks Peninsula Loess. As water contents increase from 6% cohesion increases to a maximum value and then decreases, whilst for angle of internal friction there is a linear decrease over the entire range of water contents, for example 52° at 6% to 13° at 23% for Stonehaven Subdivision loess-colluvium. Maximum values for primary airfall (in situ) Port Hills loess fill and colluvium the cohesion maximum was approximately 210 kPa at 10% water content. Friction angle trends were quite similar for all sites.

A secondary aim of this thesis was to examine lime stabilising effects on compacted loess fill. Trials were carried out at the Whaka Terrace field site where excavated loess fill from a test pit was mixed with hydrated lime at 2% by weight of the total soil mass, and then

compacted back into the same test pit so that the treated soil could be cured under field conditions. Untreated samples were taken from the base of the test pit and treated soils were extracted from the treated compacted layers at intervals of 1 week, 1 month and 2 months after compaction. Samples were prepared at two different water contents, 15% and as "as wet as possible", so that a shear strength dependence on water could also be established.

Results suggest that 15% water content treated samples had 30-40 kPa more cohesion and 5° more angle of internal friction than the "as wet as possible" treated samples. For example after 7 days of field curing cohesion was recorded at 30.7 kPa and 4 kPa for water contents of 16.9% and 19.1% respectively, and angle of internal frictions were 30° and 23.4° for the same respective water contents. Shear strength values increased over time with a rapid rate of increase after the first week of curing, and then a slowing down thereafter. A maximum cohesion of 25 kPa and angle of internal friction was recorded for samples cured in the field for 68 days as compared to untreated loess, which had cohesion of 0 kPa and 11.5° for the angle of internal friction. Both were tested at "as wet as possible" water contents.

Future work should entail the determination of effective stress parameters c' and ϕ' by determining matric suctions (negative pore pressures) to determine if the trend seen in this project is something new or an artefact of the analysis used, and more study centred on loess from the Akaroa Harbour region.

In conclusion it has been demonstrated in this thesis that Banks Peninsula Loess shear strength has a dependency on water content in terms of total stress parameters c and by way of using the triaxial test apparatus in the unconsolidated undrained condition.

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CHAPTER 1

Introduction

1.1 Project Background

Banks Peninsula Loess is an aeolian silt deposit formed from the products of glacial grinding in the Southern Alps. It is reported to have zero cohesion and an angle of internal friction of 25-30°. Ostensibly the amount of water present in loess deposit is the controlling factor in how much strength it exhibits.

Strength of Banks Peninsula Loess, with particular emphasis on total stress shear-strength parameters cohesion (c) and angle of internal friction (ϕ), is the subject of this thesis. Cohesion and the angle of internal friction provide most of the necessary information to calculate slope stability, retaining wall design and bearing capacity. The aim of this thesis is to establish cohesion and angle of internal friction data at nominated water contents that will give engineers a much more workable solution than the conservative zero cohesion and 25-30° angle of internal friction that is currently used.

A secondary aim examines the effect that lime treatment has on loess fill shear strength parameters.

Five field locations have been chosen in order to fulfil the aims of this project with special emphasis placed on Port Hills Loess, as this is where the majority of residential development on Banks Peninsula occurs. The sites are:

a) Moncks Spur, Port Hills
b) Stonehaven Subdivision, Port Hills
c) Whaka Terrace, Port Hills (lime treatment project)

d) Worsleys Spur, Port Hills

e) Duvauchelle, Akaroa Harbour (not the Port Hills)

Locations of these sites are presented in Figure 1.1 below.

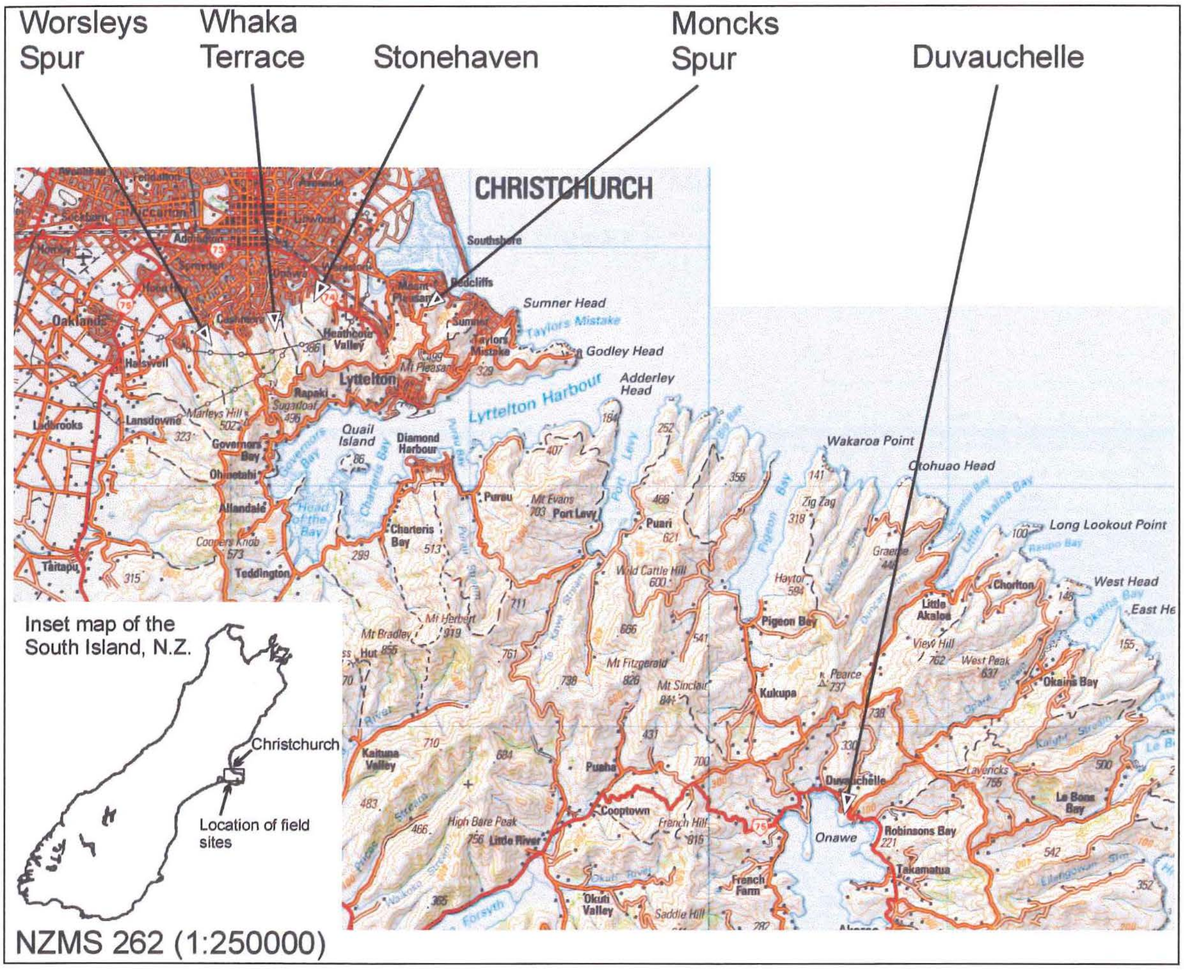


Figure 1.1 Localities of field sites on Banks Peninsula, New Zealand

1.2 Thesis aims and objectives

The overall aim of this thesis is to investigate the dependence of shear strength on water content of Banks Peninsula Loess soils. This has been achieved by:

1. Obtaining total stress shear strength parameters (cohesion and angle of internal friction) using the triaxial test method.
2. Evaluating results in terms of the total stress parameters c and ϕ.
3. Comparing cohesion and angle of internal friction to water content and degree of saturation.

A secondary aim is to determine triaxial test strength gains in uncompacted loess fill cured in the field with hydrated lime by:

1. Excavating a test pit and applying hydrated lime to the excavated material, then recompacting excavated material back into the test pit so that lime-curing can occur under "field conditions".
2. Obtaining total stress shear strength parameters cohesion and angle of internal friction of untreated and treated loess fill at two different water contents using the triaxial test method.
3. Ascertaining the strength gain over time of treated loess in terms of cohesion and angle of internal friction.

1.3 Loess terminology and origins

1.3.1 Definition

Amateur geologist Karl von Leonard first used the word loess (from the German *loesch*) in the 1820's to describe loose, friable, silty deposits along the Rhine Valley near Heidelberg. Two definitions of loess are given below; the first was obtained from the Encyclopaedia of Geomorphology as defined by Pecsi (1968):

> ...By classical definition, loess is a largely homogenous, unstratified silt. It is usually a permeable, porous, unconsolidated sediment apt to form vertical cliffs or bluffs. It is commonly yellow or buff in colour owing to its content of finely dispersed limonite, though sometimes is grey. The term of German origin meaning "loose" was used in the Rhine valley about 1821 (Scheidig, 1934) and employed by Lyell in 1934. Primarily it is aeolian, and is associated with proglacial arid climate conditions.

Pecsi (1968), however, goes on to say that this definition is no longer fully accepted; loess is not always homogenous but can be separated into "loess packets", which are a series of "cyclic members". This definition does not hold true for Banks Peninsula Loess as it is relatively non-permeable (K values less than 10^{-7} m/s).

The second definition is more appropriate for Banks Peninsula loess and is provided by Pye (1987):

> ...Loess is defined as a terrestrial wind blown silt deposit consisting chiefly of quartz, feldspar, mica, clay minerals and carbonate grains in varying proportions. Heavy minerals, phytoliths, salts and volcanic ash shards are also sometimes important constituents. In a fresh (unweathered) state, loess is typically homogenous, non- or weakly stratified and highly porous. Most commonly it is buff in colour, but may be grey, red, yellow or brown. When dry, loess has the ability to stand in vertical sections and sometimes shows a tendency to fracture along systems of vertical joints, but when saturated with water the shear strength is greatly reduced and the material is subject to subsidence, flowage and sliding. The grain-size distribution of 'typical' loess shows a pronounced mode in the range 20-40μm (5.7-5.65ϕ), and is up to 10% fine sand (>63 μm), but in cases where the sand content exceeds 20% the term sandy loess is appropriate. Up to 20%(<2μm) is not unusual in typical loess; if the sediment contains more than 20% clay it can be described as clayey loess.

However, not all of Banks Peninsula loess contains carbonate grains in varying proportions and a more suitable definition is given by Raeside (1964) in his description of South Island Loess:

> ...any fine-textured deposit of aeolian origin other than sand dunes (where particles are transported chiefly by saltation) or tephra. It thus embraces all Aeolian deposits where transport has been primarily by suspension, irrespective of content of organic matter, mineralogical composition, calcium carbonate content, degree of compaction, or texture.

This definition entitles the Banks Peninsula deposits to be classified as loess even though, against international definition, it is often non-calcareous and relatively non-permeable.

1.3.2 Occurrence

Most of the World's loess has been deposited in the Quaternary, suggesting that loess is ephemeral (easily eroded). In any case the majority of loess from around the World has been deposited predominantly during the latter stages of the Last

Glaciation (Pye, 1987). Loess and loess-like deposits cover 10 percent of the Earths land surface, of which half can be subdivided into the air fall deposit and the other half reworked or redeposited loess formed from the erosion of the former. The most extensive blankets of loess (Figure 1.2) are found in China, Central Asia, Ukraine, Siberia, central Europe, Argentina and the Great Plains of North America, with the thickest sequence measuring 355m from the loess Plateau of China (Pye, 1984). Loess covers a variety of relief forms including river terraces, pediments, alluvial fans and steep mountain slopes up to 40^{o} (Pye, 1984).

Pecsi (1968) states simplistically that finer particles are more prevalent upslope and further from source due to the decreased load bearing capacity of prevailing winds, which is certainly seen in the deposits of Banks Peninsula, although topography is also a factor in this case.

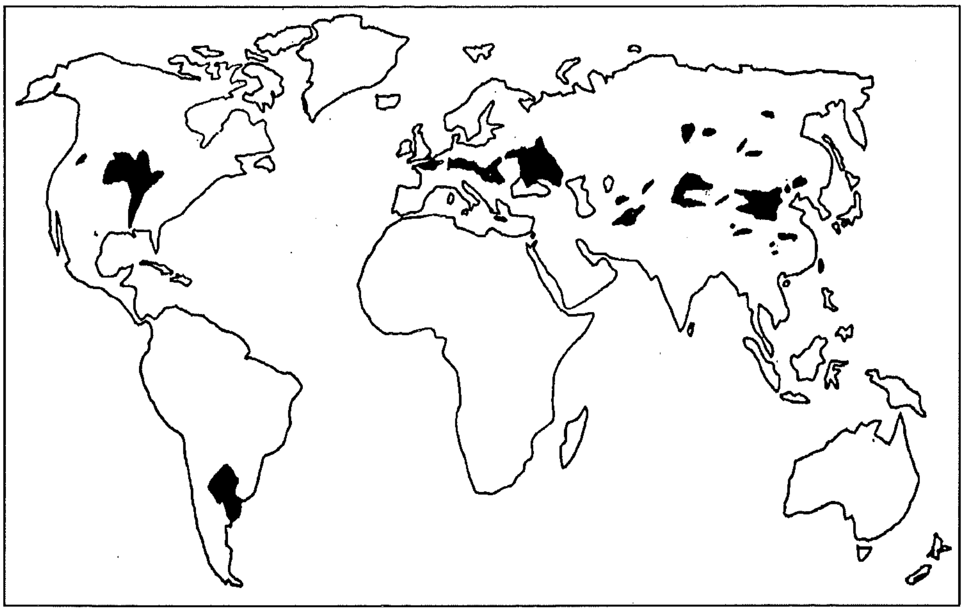


Figure 1.2 Global distribution of loess (from Pye, 1987)

1.3.3 Loess Formation

Many theories of loess formation and deposition were offered in the 1800's and early 1900's, they include: fluvial, lacustrine, marine, volcanic, cosmic, animal and vegetable matter, weathering and pedogenisis of a parent material.

However, Smalley (1966), in Pye (1987), provides an acceptable description for loess formation, which is: 1) formation of loess-sized particles (predominantly silt sized); 2) transport on prevailing winds of this material; 3) air-fall deposition; and 4) post depositional modification. Pye (1987) goes on to say that the key requirements for a high sustained rate of aeolian dust transport are: 1) the existence of bare unstable geomorphic surfaces composed of poorly sorted sediments with a high silt to clay ratio; and 2) a fairly high frequency of strong turbulent winds. In the case of Banks Peninsula Loess, the above two points are represented by fluvio-glacial outwash fans and the prevailing North-Westerly winds.

1.3.4 Source and origin of Loess

Ferdinand von Richthofen (1870) was the first to propose the theory of aeolian deposition, but even until 1944 opponents of this theory still existed. Russell (1944) was the last to present a theory other than aeolian; he proposed an in situ theory of formation in which alluvium deposited on a floodplain undergoes pedogenesis to produce parent loess material (Loessification). Pye (1984) summarises the main lines of evidence that support an aeolin origin for air fall loess:

1) loess typically forms a blanket over a variety of relief forms and extends over a wide altitudinal range
2) the thickness of loess and mean size of silt varies inversely with distance from dust source
3) zones of loess and associated deposits of wind blown sand often show clear geographical relation to the prevailing winds
4) loess is unstratified and free of pebble stringers which are common in subaqueous deposits
5) the mineralogical composition of loess often is quite different from that of the underlying rock or sedimentary formations, making loessification origin unlikely
6) Modern day accretion of loess by deposition of wind blown dust can be observed in some areas, for example Alaska and the northern Negev dessert

7) The grain size characteristics of modern dusts are very similar to those of Pleistocene loess

Most experts would agree that loess deposits in Europe and North America have a glacial origin (indeed this is also correct for South Island loess), however, the source of loess deposited in China and Central Asia is not so clear. Pye (1987) asserts that loess of Central Asia and China has a desert origin, whilst Smalley and Krinsley (1978) suggest that although loess was blown from the desert it was certainly not formed there, advocating that glacial-ground loess was transported fluvially from an adjacent mountain range, deposited in the desert, and eventually blown back to its present location. Pye (1987) refutes this, stating that there is now clear evidence that loess-sized particles can be formed in deserts due to the combined action of frost and salt weathering.

At this point, it is prudent to point out that the first proponent of the glacial origin theory was New Zealander, John Hardcastle, in 1890 (Smalley, 2001):

> ...The source of the dust. –There was only one source possible in these latitudes for such a quantity of dust, and a mere hint as to its nature will suffice. If we consider the loess to belong to the great Ice Age there will be no difficulty. The dust was 'rock-meal', produced by the great ice mill, and spread out by rivers of sludge for the winds to dry, pick up, and bear away, losing more or less of their load whenever they passed over a vegetated region... No other agent than ice could have produced so great a quantity of fine material.

With this observation in mind, Hardcastle then went on to suggest a link between climate and loess deposition, which was also a first for loess research. However, his observations went largely unnoticed and suffered what Smalley (2001) termed the 'Mendal effect', which is where a major discovery is made but is not noticed by the rest of the world. In effect, Pavel Tutkovskii (1899) was the first to bring the glacial origin theory to the international arena of loess research, but some ten years after Hardcastle's comments.

1.4 New Zealand Loess deposits

1.4.1 Loess types and origins

In his review of Loess, Pye (1987) subdivides New Zealand Loess into two distinct areas, the North Island and the South Island. This is because North Island Loess is thought only to have had a cold weathering origin of tephra, whereas South Island Loess has been formed from glacial grinding and cold weathering of greywackes and metamorphics of the Southern Alps. He states that there have been 5 distinct periods of loess accumulation in the North Island separated by paleosols and/or ash layers, with the youngest being 20600yr BP (dated from a layer of tephra), and 6 distinct phases in the South Island also separated by paleosols with the youngest being 9900-11800. The above date for South Island loess was provided by Tonkin et al (1974) from radio carbon dating of Timaru Downs loess, but they supplied no dating controls. Tonkin et al (1987) do however conclude that periods of loess deposition coincide with conditions of glacial retreat.

The first to classify N.Z. aeolian silt deposits as loess was Sir Julius von Haast (1879) in his description of yellow Canterbury silts (Raeside, 1964). Hardcastle (1889) reaffirmed this view to explain the origin of loess on the Timaru Plateau. Some 75 years later, Raeside (1964) reviewed the loess of the South Island and gives the first notable definition of loess as mentioned above. He advocates that South Island Loess be split into three zones (Marlborough and Canterbury, Otago, Southland) each differentiated by their geographically different source areas (although still the same origin i.e. glacial grinding in mountainous terrain). Loess from Southland is sourced from tuffaceous greywackes, Otago Loess from metamorphic schists, and Canterbury/Marlborough loess from Southern Alps greywackes. It appears that Raeside (1964) is the first to present the idea of loess accumulation during glacial retreat, not glacial advance as once thought, however, he suggests that only tentative correlations be made with other Pleistocene events until a definite chronology can be ascertained.

Further subdivision of Canterbury Loess by Ives (1973) divides airfall loess into two types, to which he gives the names “post-stadial loess” and “interstadial loess". The first loess type is the thicker loess deposits seen on the Timaru downlands and Banks Peninsula; he calls them "post-stadial loess". Ives (1973) suggests that this loess type was derived from pre-existing loess deposits located on fan surfaces of the Canterbury Plains, and then relocated to their present position, which he thinks took some 2000 years to complete based on the dating Tonkin (1974) provides (9900-11800 B.P.) for this loess type. The second type of loess Ives (1973) calls "interstadial loess", which is loess that has been deposited on fan surfaces adjacent to major rivers crossing the plains; he says that Recent loess or post-glacial loess is an example of this. A flow chart of loess depositional history has been synthesised in Figure 1.3 from information given by Tonkin et al (1974) and Ives (1973).

	Present Day
Post glacial Intergalcial conditions	↑ Soil formation
Major glacial recession	Loess accumulation 11800-9900 BP
Stadial conditions multiple glacial advances	↑ Soil formation
	Loess deposition before 31000 BP

Figure 1.3 Flow chart of recent loess depositional history (synthesised from Tonkin et al, 1974 and Ives 1973)

Although the relocation theory is just that, what is conclusive is that the majority of loess deposition in the South Island accompanied glaciation of the Southern Alps in the Late Pleistocene and probably occurred at the time of glacial retreat (Raeside, 1964; Tonkin, 1974; Bell and Trangmar, 1987).

1.4.2 Banks Peninsula Loess

Just sixteen years prior to Raeside's work, Sparrow (1948) submitted his thesis on the Loess of Banks Peninsula. His work deals mainly with the topic of formation and provides "negative" and "positive" evidence to the assertion that Banks Peninsula Loess was in fact aeolian in origin. It is pertinent to point out that only four years before this investigation, researchers were still advocating an in situ model of loess formation (Russel, 1944). Sparrow subdivides loess into: 1) primary or typical loess; 2) secondary or redeposited loess, coincidently, as Bell and Trangmar (1987) do some 40 years later. However, an inconsistency in Sparrow's thinking has him using Russell's definition for loess, which includes qualities such as unstratified, calcareous and porous. Clearly, this is not the case and it seems unusual that a definition be adopted from someone who is advocating a theory of origin quite different from his own.

From a purely pedological point of view Griffiths (1973) presents the most extensive review of Banks Peninsula Loess. Various sections throughout the two calderas are discussed and logged, and with this information he concludes that loess can be separated into two distinct types of the same facies (Figure 1.4). The first is Birdlings Flat Loess, which is found on the lower slopes of the Lyttelton

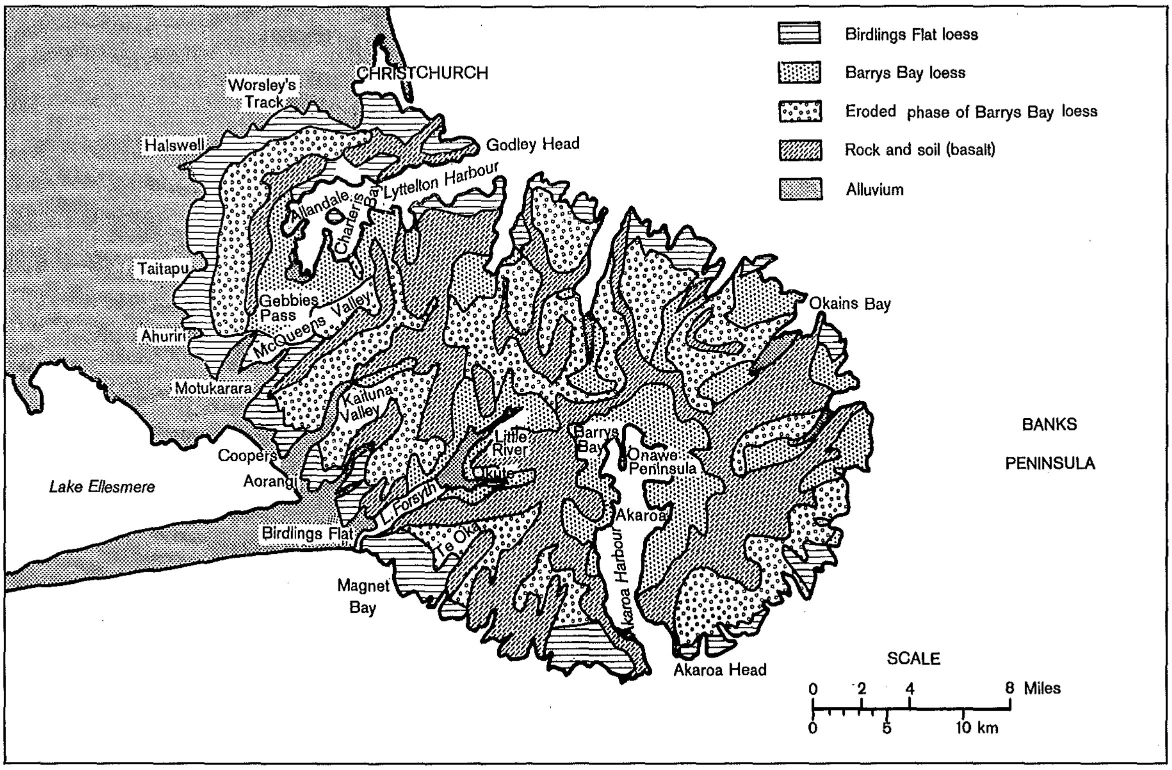


Figure 1.4 Banks Peninsula Loess distribution (from Griffiths, 1973)

caldera; it is calcareous, sandy, somewhat stratified, and is separated into two major layers by one paleosol and a layer of loess colluvium. The second is Barrys Bay Loess, which is found on the headwaters of all harbours and inlets of both calderas; it is silty, non-calcareous, less stratified than Birdlings Flat Loess, and is separated into 4 major layers by three paleosols. A date for the first paleosol in Barrys Bay loess is given at 17450+/- 2070 yrs b.p., which is hardly post-stadial. This differs markedly with the age that Tonkin (1974) provides, and demonstrates that the loess paleoclimatic problem is very complex. Both soils are described using soil horizon nomenclature compatible to that published by Clayden and Hewitt, (1989), with A, B, Bx, C and Cx horizons layered downward, roughly in that order.

Another layering model for Banks Peninsula Loess, which is more descriptive rather than genetic, has been established by Hughes, (1970), and has since been adopted by most of the engineering profession. Hughes (1970) argues that to avoid the problem of deciding to what extent the soil horizons, especially the hardpan, are a result of, or influenced by current pedological processes, the soil be divided into three different layers: 1) the surface soil layer; 2) the compacted layer; and 3) the parent material loess layer. All are outlined in Figure 1.5 below.

The last major piece of research carried out on Banks Peninsula Loess (specifically Port Hills Loess), aside from thesis work, is accredited to Bell and Trangmar (1987). They summarise Port Hills regolith deposits into five main types, which are: 1) in situ (primary air fall) loess; 2) loess colluvium; 3) mixed deposits of loess and volcanic derived colluvium; 4) volcanic-colluvium 5) residual regoliths, derived from the in situ weathering of volcanic rocks on erosion surfaces. Six erosional processes are also reviewed, which are summarised, as with soil types in Figure 1.6. Remedial measures for erosional processes (including mass movements) creating geotechnical problems that affect residential development on the Port Hills are also discussed and will be further reviewed in the discussion chapter.

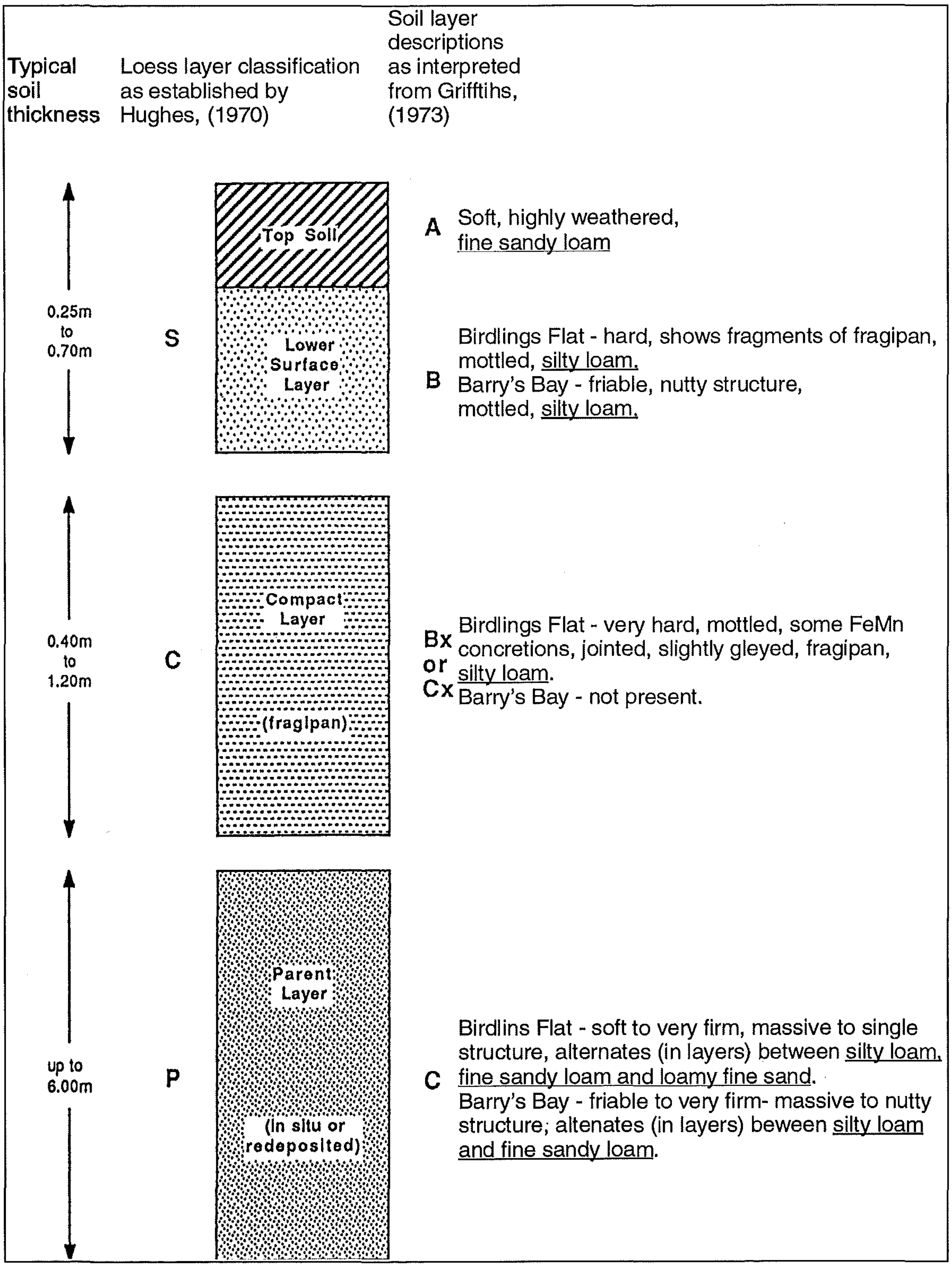


Figure 1.5 Banks Peninsula in situ Loess layering (modified from Hughes (1970) and Goldwater (1990))

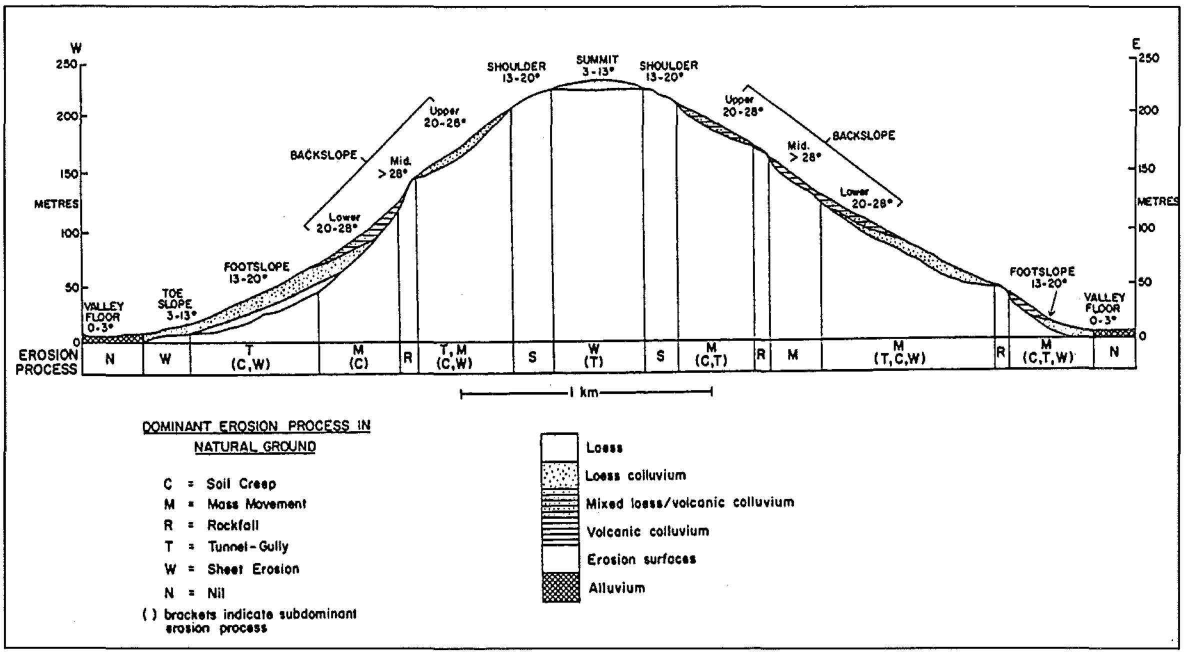


Figure 1.6 Port hills loess classification and erosional processes (from Bell and Trangmar, 1987)

1.5 Project setting and thesis methodology

1.5.1 Geological setting

Banks Peninsula lies on the east coast of the South Island, New Zealand, has an area of roughly 1200 km^2, and the highest point (Mt Herbert) is 920m above sea level. The Peninsula was once an island, but has since been connected to the mainland by outwash fans spreading from the Southern Alps (Liggert and Gregg 1965). Stipp and McDougall (1968) thought that volcanic activity ceased before the Kaikoura orogeny, as no folding is present in the basement strata at Gebbes Pass, although they say this could be the result of Banks Peninsulas position on the west end of the Chatham Rise, which is too distant to be affected by mountain building processes.

Two periods characterise Banks Peninsula Geology: Pre-Miocene and Miocene (Weaver and Sewell 1986). The Pre-Miocene geology consists of rocks forming the basement high underneath Gebbies pass, and include: 1) Torlesse sandstones and mudstones, which are Triassic in age and outcrop in the Gebbies Pass area; 2) Two-pyroxene andesites and peraluminous, high silica rhyolites of the

McQueens Volcanics, they are thought to be mid-Cretaceous in age and also outcrop in Gebbies Pass; and 3) Charteris Bay Sandstone, which is a Tertiary sedimentary unit and overlies coal measures that are late Cretaceous in age (Weaver and Sewell 1986).

Six main periods of volcanic activity have been documented in the Miocene Banks Peninsula geology, as follows: 1) Governors Bay volcanics, which were erupted between 12-11 Ma, outcrop at the headwaters of Lyttelton Harbour, and are composed of andesites and rhyolites; 2) Lyttelton Volcanics, which were erupted between 11-10 Ma from centres south and west of Quail Island, and consist of plagioclase-clinopyroxene-olivine-phyric hawaiites, mugearites, benmoreites and trachytes; 3) Mt Herbert Volcanics; erupted between 9.7-8.0 Ma, and are composed of hawaiites, mugearites and some basalts; 4) Akaroa volcanics, which were erupted between 9.0–8.0 Ma and range from olivine basalts to trachytes; 5) Church Volcanics, which were erupted 8.1-7.3 Ma indicating renewed activity in the Lyttelton caldera, and are composed of basanitoids and alkali olivine basalts; 6) Stoddart Volcanics, which were erupted between 7.0-5.8 Ma, and consist of olivine basalts and olivine hawaiites (Weaver and Sewell 1986). Distribution of Banks Peninsula volcanics are shown in Figure 1.7.

Both calderas were then sea breached due to erosion, decreasing both volcano heights by at least a half. The majority of erosion was thought to occur over a period of 1.5 to 2 million years in the Miocene, with some increases in erosion occurring during the Quaternary as a result of sea-level fluctuations due to glaciation (Bell and Trangmar, 1987).

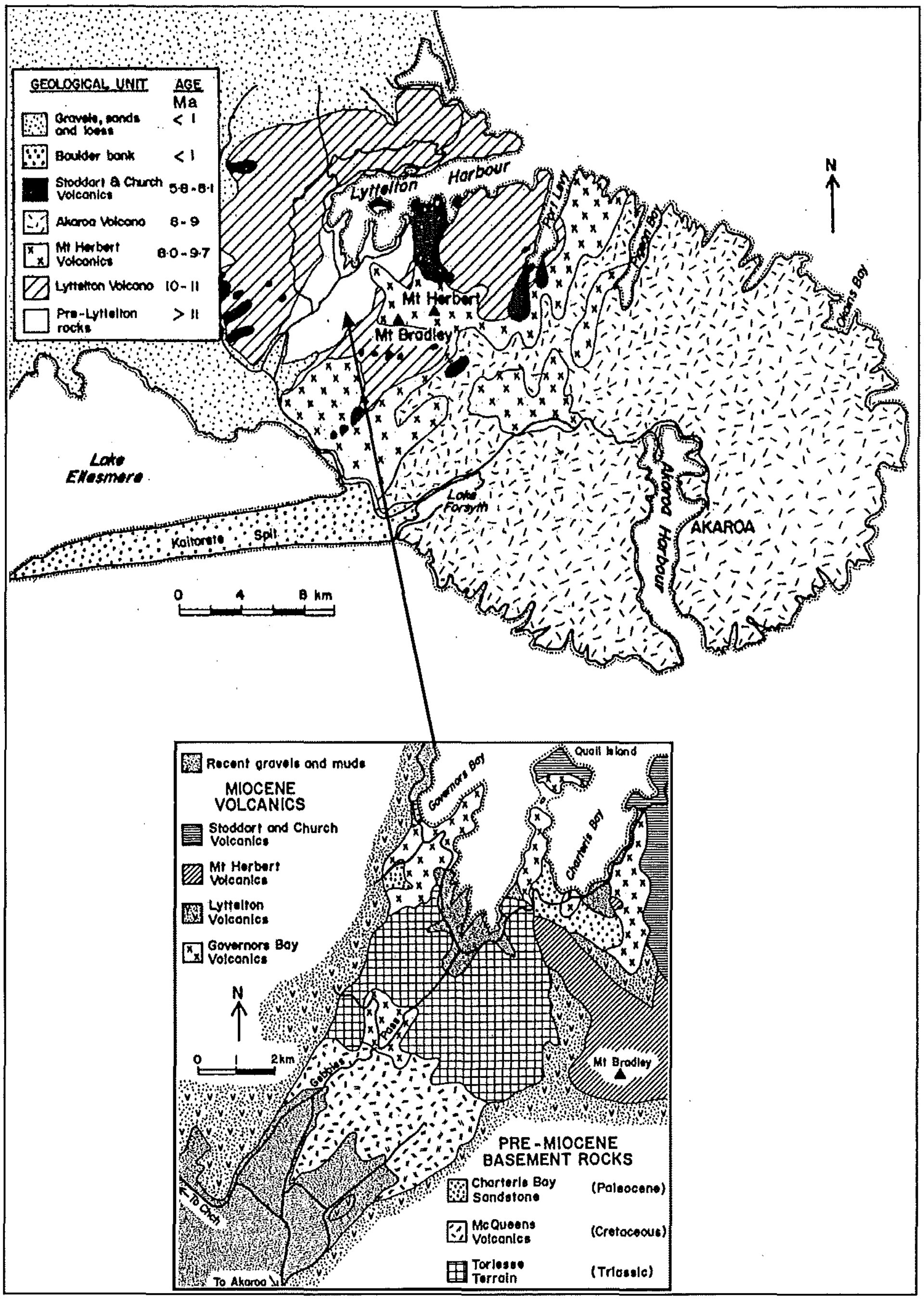


Figure 1.7 Distribution of Banks Peninsula volcanics (from Weaver and Sewell, 1986)

1.5.2 Topography and climate

Topography of Banks Peninsula is generally hilly, with many steep (greater than 20^{o}) slopes on the inner walls of the calderas, and hilly to rolling on the outer slopes of the calderas; lava flows tend to form benches along the ridges (Grifftihs, 1973). Climate along the outer edges and lower elavations of both calderas is subhumid, with an average rainfall of about 700mm per year, whilst at higher elevations and towards the centre of the calderas the weather is much wetter and can receive up to 1400mm of rain per year. The majority of sites selected were in the drier subhumid environment.

1.5.3 Site selection criteria

Sites were selected with geographical variation in mind; however, cost of excavation for sampling purposes limited the number of potential sites; subdivisions and residential developments undergoing construction were preferred. Timing of sampling and the digging of test pits was constrained to the drier months. The site chosen for the lime treatment project had to have experienced some instability problems in the past and preferably this would have to be a site covered in uncompacted fill.

1.5.4 Project methodology

Aims and objectives have been achieved by carrying out the following:

1. Selecting a representative number of sites on Banks Peninsula for the collection of triaxial tube samples, and index testing of samples.
2. Carry out Engineering Geological logging of sample sites and pits.
3. Preparation of samples in such a way that the water content can be controlled so that triaxial tests of samples can be conducted in separate water content groups.
4. Laboratory testing of samples for cohesion and angle of internal friction using the undrained unconsolidated triaxial test without measurement of pore pressure.
5. Analysis of results to compare shear strength and water content.
6.

For the subsidiary project of the lime-stabilised shear key aims and objectives were fulfilled by:

1. Selection of a potentially unstable uncompacted loess fill slope.
2. Surveying of slope dimensions by theodolite and determining the buried soil horizon using the Dynamic Cone Penetrometer, hand auger and test pit.

3. Obtaining shear strength parameters of untreated loess fill from that site.
4. Conducting a field trial for loess lime treatment and curing by excavating a pit, mixing the excavated soil with lime, placing it back into the pit compacted, and obtaining samples at one week, one month and two months to be tested for treated shear strength parameters.
5. Using those treated shear strength parameters to computer model a lime-stabilised shear key.

1.6 Thesis Format.

A brief outline of the topics covered in this thesis is presented below:

- Chapter 2 presents the argument in detail, discussing details of loess shear strength and methods used to determine it. All relevant literature has been reviewed.
- Chapter 3 details site descriptions and test pit logs for all field sites excluding the Whaka Terrace site.
- Chapter 4 presents the results obtained from triaxial testing for all sites described in Chapter 3 as well as results for untreated Whaka Terrace Loess. Trends will be explored from data analysis and then a discussion made as to the legitimacy of stress parameters (c and ϕ) for Banks Peninsula Loess
- Chapter 5 focuses on the effect of lime treatment on Whaka Terrace loess fill, and presents the results (in terms of factor of safety) of a computer modelled shear key at the base of the Whaka Terrace Loess fill slope. A small review of literature concerning lime treatment of Port Hills loess precedes the results section.
- Chapter 7 summarises principals and conclusions and presents ideas for further research.

CHAPTER 2

Literature Review and Test methods

2.1 Introduction

Determining the shear strength of Banks Peninsula loess has been a difficult task to achieve, and although most engineering professionals use zero cohesion there is some cohesion present, either in the form of soil suction or clay minerals. Most agree upon an average figure of about 25°-30° for the angle of internal friction. Past researchers have reported an inverse relationship between water and shear strength, but it has not been looked at in detail. This chapter explains how this study will attempt to determine the relationship between shear strength and water content of Banks Peninsula Loess by presenting and applying the fundamentals of shear strength analysis, and by reviewing and applying past literature (both international and local).

2.2 Shear strength of soils

2.2.1 Basic concepts

In the calculations required for the stability of a slope, retaining walls and/or the bearing capacity of a foundation many parameters are required. They include soil unit weight, pore pressure, slope geometry and shear strength. Of these shear strength is the most difficult of all parameters to obtain, because of the complicated nature of laboratory test methods, particularly the analysis of test results, sample preparation and relation to in situ behaviour.

Shear strength is essentially the amount of shear stress (a force per unit area which acts tangential to a surface) a soil can sustain before failure (defined below) occurs. Thus the shear strength of a soil, coupled with a particular method of analysis, will determine the maximum or ultimate (failure) load that can be applied to a foundation resting on soil or the ultimate force required to cause failure of a soil mass forming a slope (Barnes, 1995). Barnes (1995) presents a simplified diagram (Figure 2.1) accompanying the definition above, although it ignores the area over which the force can act.

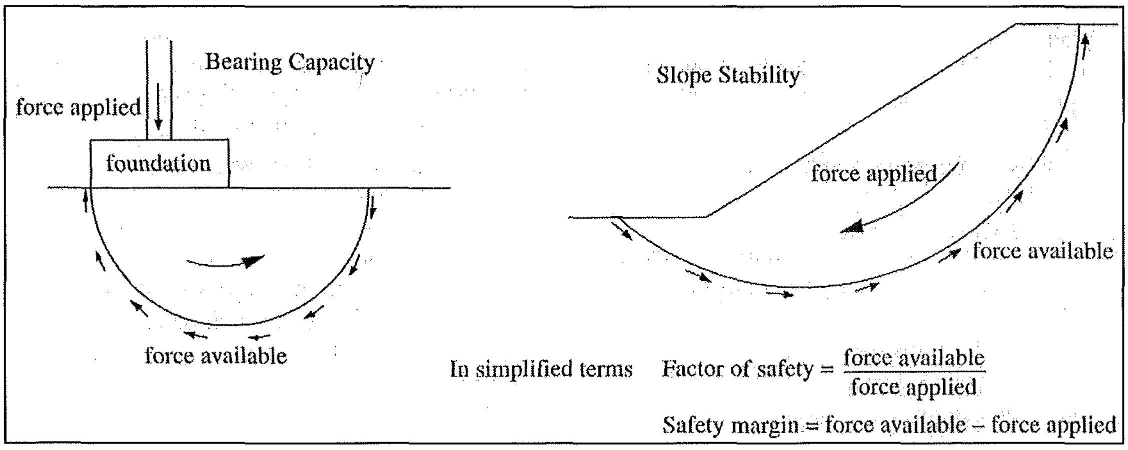


Figure 2.1 Soil shear strength as applied to foundations and slopes (from Barnes, 1995; Figure 7.1)

Failure can be defined in different ways, and it is important to outline which failure type is important to a specific engineering problem. Barnes (1995) gives five definitions (as well as an accompanying diagram, Figure 2.2) of soil failure:

Yield: Although not the maximum shear stress available, if the soil is stressed any further beyond the point Y in figure 2.2 the strains and movements of the earth (foundation, slope, etc) could be so large and irrecoverable that they may be deemed to have failed as a serviceability limit state. τ_y represents a yield stress.

Peak shear strength: This is the maximum shear stress which can be sustained. It may be dangerous to rely on this value for some brittle soils due to the rapid loss of strength that occurs when the soil is strained beyond this point.

Ultimate strength: For loose sands and soft clays work-hardening may continue to increase the shear stress that can be sustained even at very large strains, so a maximum stress is not achieved. A maximum strain limit must then be imposed, usually related to the performance of the earth structure, say 10 to 20% strain such as point U in figure 2.2.

Critical state strength: This is sometimes referred to as the ultimate strength. After a considerable amount of shear strain a soil will achieve a constant volume state (by the soil structure expanding or contracting) and it will continue to shear at this constant volume without change in volume or void ratio. These shear strains must be uniform throughout the soil and not localized. It is sometimes referred to as the constant volume strength (ϕ_{cv}).

Residual strength: This is also sometimes referred to as the ultimate strength. After a considerable amount of strain on a single slip zone or surface (point R in figure 2.2) the particles on each side of this surface will rearrange to produce the lowest possible or residual strength. This strength is important in the re-activation of old landsides and is obviously more significant for platy minerals such as clays.

Although the stress paths below in figure 2.2 do not represent silty soils (loess), dry silty soils would follow the path of a stiff clay, and wet silty soils would follow the path of a soft clay.

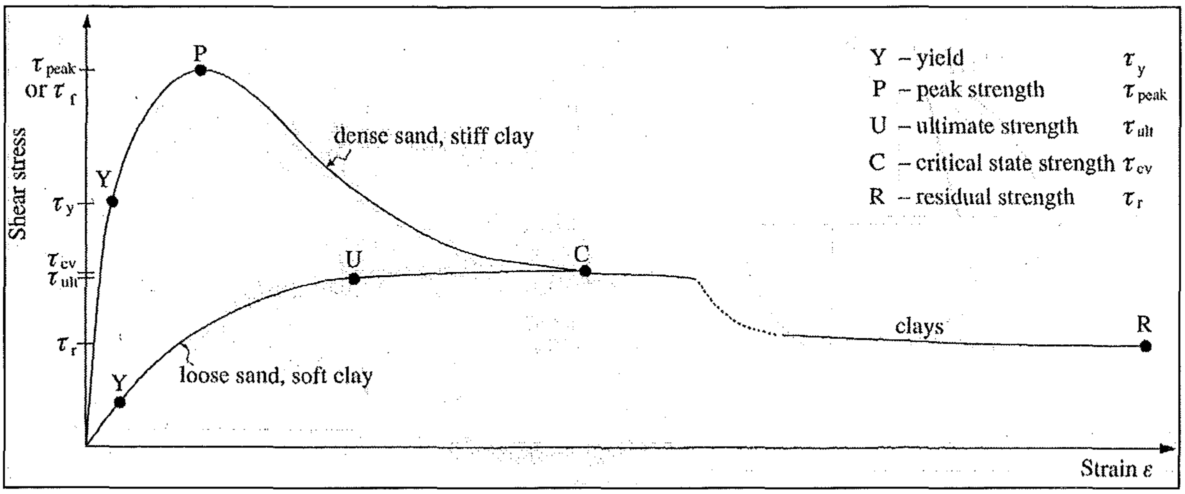


Figure 2.2 Idealised stress/strain paths for soils (from Barnes, 1995; Figure 7.11).

The source of soil shear strength is dependant on the type of soil present. Non-cohesive soils obtain their shear strength through friction created on the surfaces of each individual particle and by particle interlock, hence their strength is reported as an angle of internal friction (or shearing resistance); particle shape strongly influences the amount of strength exhibited in a non-cohesive soil. Cohesive soils also gain some of their shear strength by internal friction, but more importantly through inter-particle bonding. These bonds can be created through various agents, the most common of which are clay bonding and soil suction effects created by negative pore water pressures. Cohesion is reported in kPa and internal friction are reported as an angle, ϕ (degrees). Shear strength in loess is a combination of clay mineral cohesion, soil suction and internal friction. Finding out exactly how much shear strength is attributed to clay bonding or soil suction is not the subject of this thesis, and would require a thesis in itself. The combination of both is termed apparent cohesion, and it is this type of bonding that this thesis is particularly concerned with, and for the purposes of this thesis both terms are interchangeable.

Stress can be represented graphically in soil using the principal stresses $\sigma 1$, $\sigma 2$ and $\sigma 3$, which act perpendicular to a three dimensional soil element along the x, y and z axes respectively. Planes on which the principal stresses act are known as principal planes, and shear stresses that act on these principal planes are zero. If stresses on two principal axes are equal (e.g. $\sigma 2 = \sigma 3$), then a condition known as axial symmetry occurs. Under this condition stresses on any soil element can be represented in two dimensions. Graphical representation of stresses in two dimensions can be plotted as Mohr circles of stress; $\sigma 1$-$\sigma 3/2$ is the circle diameter and shear stress, whereas $\sigma 1$+$\sigma 3/2$ is the centre of the Mohr circle on a shear stress-normal stress plot (t-s plot) (Figure 2.3).

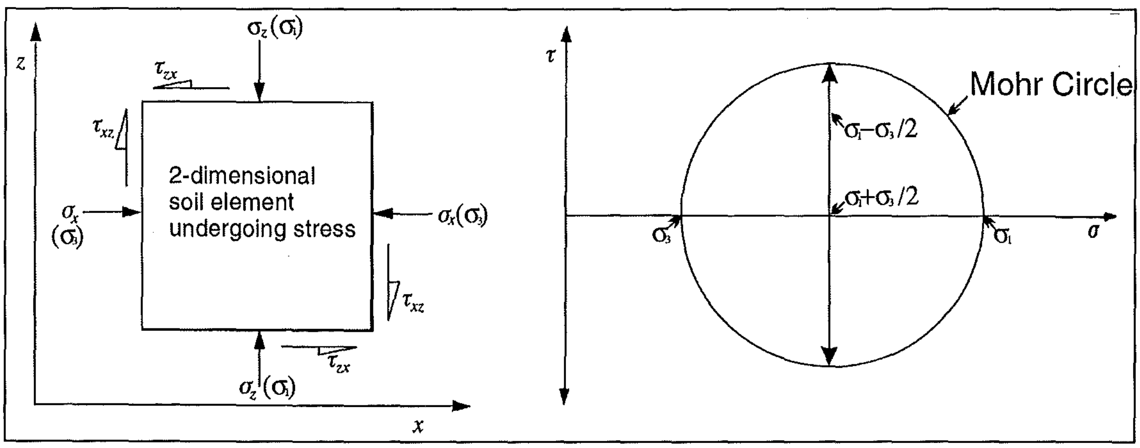


Figure 2.3 Stresses represented in two dimensions (modified from Barnes,1995)

2.2.2 Shear strength analysis

2.2.2.1 Analysis of all soils: the total stress state.

In determining the shear strength of a soil present under a building or within a slope, manmade or otherwise, it has been common practice to use the Mohr-Coulomb (equation 2.1) failure criterion, which is defined as:

$$\boxed{S_{(f)} = c + \sigma_n \tan \phi} \qquad (2.1)$$

where

$S_{(f)}$ = shear strength at failure (kPa)

c = cohesion, total stress parameter (kPa) (plastic soils only)

σ_n = normal force applied i.e. the weight of a bearing load or gravity acting on a slope. This represents the total stresses acting on a soil (kPa)

ϕ = the internal shearing resistance, total stress parameter (degrees)

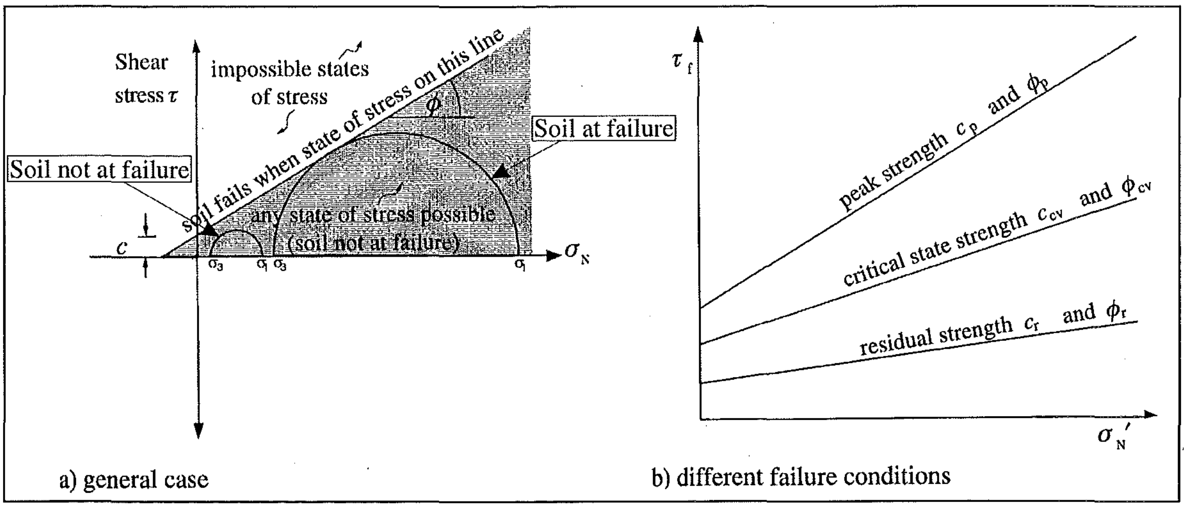


Figure 2.4 Mohr-Coulomb failure condition (modified from Barnes, 1995)

Peak shear strength data ($\sigma 1$ and $\sigma 3$) obtained from triaxial testing can be plotted as Mohr circles representing normal stresses applied to the sample; angle of internal friction is the angle of the failure envelope tangential to the circle, whilst cohesion is the intercept of the failure envelope on the shear strength axis (Figure 2.4).

2.2.2.2 Analysis of saturated soils: the effective stress state

With the addition of water compressive strength is affected, and in terms of a saturated soil the effect will be to reduce the shear strength. Terzaghi (1936) was the first to define a relation between loading and moisture content (equation 2.2):

$$\boxed{\sigma' = \sigma - u_w} \qquad (2.2)$$

where u_w is the pore water pressure, σ' is the effective normal stress, and σ is the total stress. As a result the Mohr-Coulomb criterion (equation 2.3) is modified to

$$\boxed{s = c' + (\sigma - u_w) \tan\phi'} \qquad (2.3)$$

Cohesion and angle of internal friction now become the effective stress parameters c' and ϕ', which provide the necessary information to calculate soil shear strength under saturated conditions It is graphically demonstrated in figure 2.5.

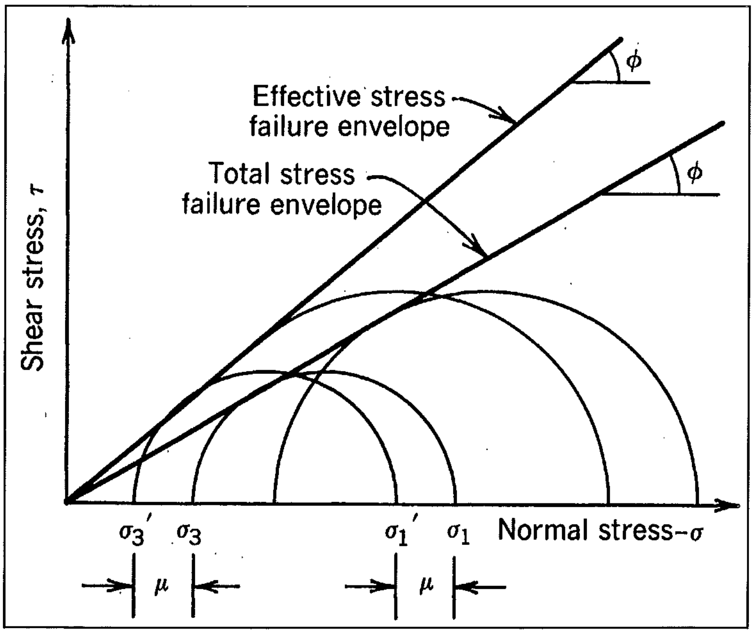


Figure 2.5 Pore pressure effects on Mohr circles of stress and failure envelopes (From Johnson and Degraff, 1988).

2.2.2.3 Partly saturated soils: matric suction

Complexity arises with the calculation of shear strength parameters in a partially saturated cohesive soil; it may be important to obtain partially saturated soil strength parameters if failure occurs before saturation is reached. Fredlund (1978) proposes a model for the determination of shear strength parameters in a partially saturated soil according to equation 2.4:

$$\boxed{S = c' + (\sigma - u_a) \tan\phi' + (u_a - u_w) \tan\phi^b} \qquad (2.4)$$

where

> "ϕ^b is the angle indicating the rate of increase in shear strength relative to a change in matric suction, (u_a-u_w), when using (σ_n-u_a) and (u_a-u_w) as the two state variables; and ϕ' is the angle indicating the rate of increase in shear strength with respect to net normal stress, (σ_n-u_w), when using (σ_n-u_w) and (u_a-u_w) as the two state variables." (Fredlund, 1978).

2.2.3 Triaxial test method

The triaxial test method and apparatus (figure 2.6), developed in the late 1930's, has the ability to impart shear stress in a soil using normal forces. It does this by enclosing cylinder of soil in a rubber membrane, applying a constant water pressure (σ_3) to the sides of the soil cylinder, and applying a normal force (σ_1) to both ends using a motorised drive at one end and a piston locked in place at the other. Strain is applied at a constant rate (which is dependent on the failure model) through the motorised drive underneath the sample, and the stress is measured by a load cell which is attached to the locked piston at the top of the sample. The shear strength of a soil is then obtained by graphing σ_1 and σ_3 on a Mohr circle plot, and applying one of the analysis methods that has been outlined above to get cohesion and angle of internal friction parameters. Porous discs placed against both ends of the sample facilitate pore pressure measurement within the sample for effective stress analysis; modified porous discs can also be used to measure negative pore water pressures (matric suction). Samples for testing have to have a height diameter ratio exceeding 2:1 so that failure planes have enough length (of sample) to develop.

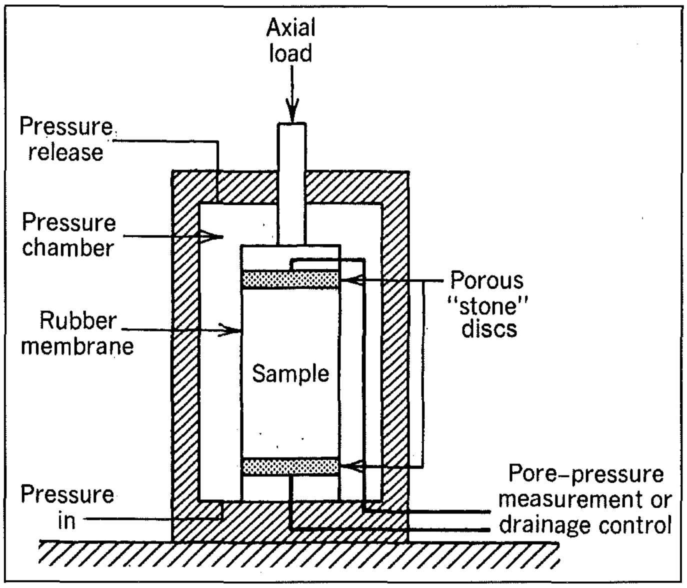


Figure 2.6 The triaxial test apparatus (from Johnson and Degraff, 1988)

There are three different types of in situ behaviour that the triaxial test can model, they are:

Consolidated-drained test (CD): The soil is consolidated using the cell pressure σ_3 and is conducted at a very slow rate against porous discs to let pore pressures dissipate so that no pore pressures develop within the soil. Because no pore pressures develop, effective stresses are known throughout the test. This test models failure in sand and soil failures where drainage occurs

Consolidated-undrained test (CU): The soil is consolidated using cell pressure σ_3 but a faster compression rate is used and no drainage is allowed causing pore pressures in the soil to rise. Pore pressures are measured to calculate effective stresses. This test models undrained failure in consolidated soils

Unconsolidated-undrained test (UU): No consolidation is allowed to happen and subsequently pore pressures build up in the soil with application of cell pressure. Pore pressures increase again, which are measured to calculate effective stress parameters; if no pore pressure is measured then the test can only be interpreted in the total stress state. This test models rapid failure in unconsolidated soils such as slope failure under heavy rainfall and the early stages of soil loading in the placement of a foundation

2.3 International literature on loess

Most international literature concerned with the mechanical/geotechnical aspects of loess centres on its ability to collapse when wetted then loaded. However, large settlements are not indicative of New Zealand loess and have not been reported.

Although geotechnical characteristics invariably vary with location, there are some commonalities evident in the literature. These are: 1) zero cohesion at saturation, 2) an inverse relation of water contents to shear strength (i.e. shear strength is relatively high when loess is dry); and 3) the resultant ability to stand in vertical cut-slopes when unsaturated. Banks Peninsula Loess is no exception in all three characteristics.

2.3.1 Asian loess

Kie (1988) presents a detailed study of loess mechanical properties from China's loess plateau, specifically Kansu province. Studies were carried out in response to the Chinese government's plan to make Lanzhou City (figure 2.7) in Kansu province a major industrial centre. Plasticity index values reported are between 10-14, and liquid limits range from 27-30 percent.

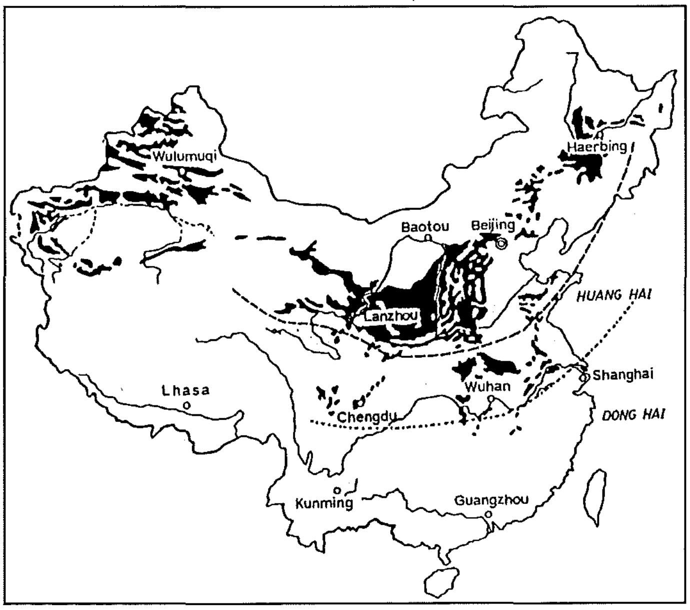


Figure 2.7 Location of Lanzhou City, China; blackened areas represent loess deposits (from Kie, 1988)

Kie suggests that the low plasticity index values are indicative of soil mechanical and physico-chemical properties, which are sensitive to a change in overall water content. Also reported is an interpretation of microstructure and a grainsize analysis. Microscope analysis reveals an odd three-dimensional structure in which large grains (50-100μm) are separated by a filler material containing particles of 5-50 microns and an embedded mass of less than 5 microns, which is made up of clay particles and soluble salts (figure 2.8). Calculations made from grainsize analysis give a percentage of 7.5-12.7 percent for the size fraction less than 5 microns (none are given for the size fraction <2μm). From the above information, Kie deduces that the overall mechanical and physico-chemical properties associated with soil collapse are governed by the bonds associated with the filler material, and especially by the constituents less than 5 microns. Kie claims that the bonding material only represents 7.5 to 12.7% of the total soil.

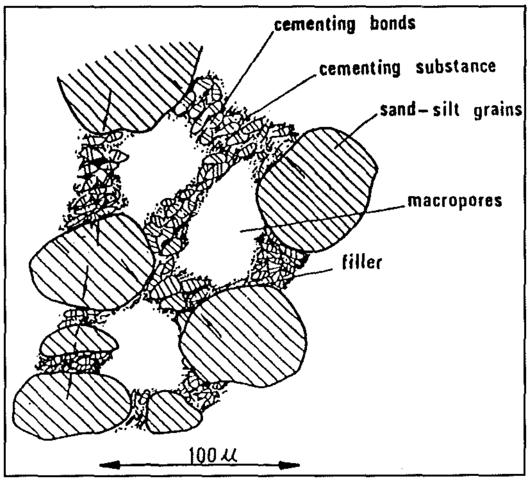


Figure 2.8 Cartoon of Lanzhou Loess microstructure (from Kie, 1988)

To asses the validity of the aforementioned propositions, Kie conducted four types of mechanical tests: 1) oedometer tests; 2) triaxial tests; 3) creep and relaxation tests; and 4) permeability tests. Only the triaxial tests will be discussed here.

Kie carried out triaxial tests on samples with a diameter of 62mm and height of 120mm. Water contents were targeted at 3, 6, 9, 15 and 17 percent, but he gives no reasons for selecting these numbers and one must postulate that they are arbitrarily chosen. The type of test is also not reported, but he does say that a hydrostatic pressure, $\sigma3$, is applied for at least two hours before testing and the testing speed is 0.36mm per minute; this would suggest that the test is a consolidated undrained test. Triaxial testing reveals a bimodal behaviour as can be seen in the size of Mohr circles in figure 2.9; after a certain threshold Mohr circle size increases substantially. Kie attributes this behavior as the superposition of two quite different mechanisms, which are

(1) the deteriorating effect of the breaking of bonds resulting in lowering of the cohesion and thus a softening of the sample;

(2) the hardening effect to the increase of mechanical friction between the granular particles within the bonds and the increase in overall contacts between the whole assembly of grains

Kie alludes to the fact that this behaviour may be associated with the collapsible nature of loess but does not attribute it explicitly.

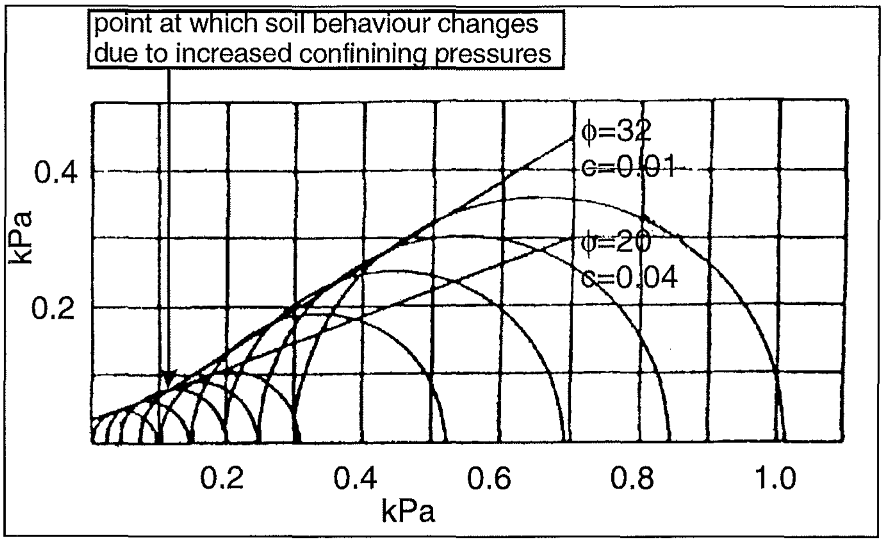


Figure 2.9 Failure Mohr's circles and envelopes for Lanzhou Loess (from Kie, 1988)

The triaxial tests were also conducted at differing water contents, however different water contents were given than those stated above and the preparation technique was not described. Only angles of internal friction were given, and they showed a definite inverse relationship dropping from 16.7° at 8% water content to 5.7° at 20% (figure 2.10).

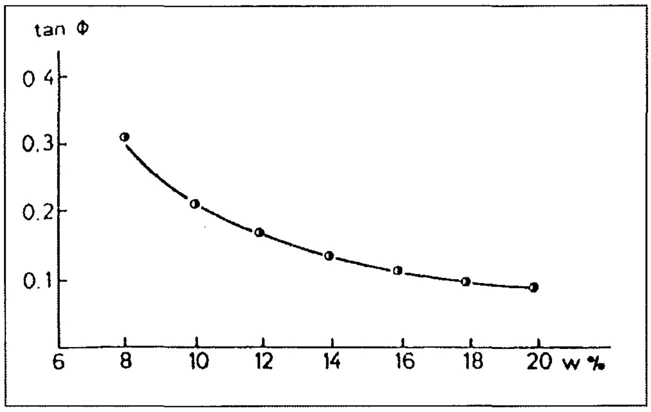


Figure 2.10 Tan ϕ as a function of water contents for Lanzhou Loess (from Kie 1988).

2.3.2 European loess

Milovic (1988) presented a paper on Loess properties from Yugoslavia concerning the collapsibility of loess, and argues that there is a correlation between dry density and initial water content based on the results from unconfined compressive strength testing. He found that unconfined compressive strength (q_u) decreased with increasing water content and that q_u increases with dry density (figure 2.11).

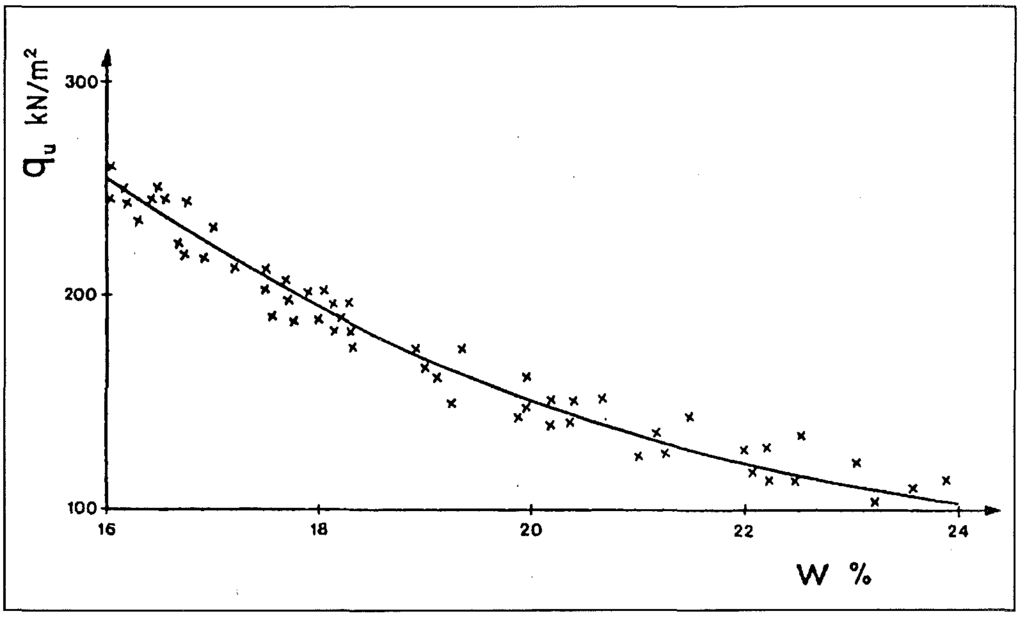


Figure 2.11 Relationship between the unconfined compression strength q_u and water content (from Milovic, 1988).

The most interesting observations made by Milovic from this project's standpoint are the comments he makes on sample disturbance. Milovic compared tests performed on piston samples with those performed on hand carved samples and concludes that sample disturbance from the piston method increases compressive strength and dry density considerably.

2.3.3 North American loess.

In a review of North American Loess and slope instability, Higgins and Modeer (1996) provide a summary of loess shear strength properties. They state that loess is characterised by a loose structure of silt and clay particles held together by a clay "binder". They also claim that this clay "binder" is responsible for most of loess' cohesion, and that this cohesion, at low moisture contents, is attributable to high negative pore water pressures (matric suction) developed in the "binder". No specific clay contents are given throughout the paper, but from grainsize distribution plots clay contents would appear to be somewhere 10 and 20%.

The first major study on North American Loess shear strength parameters, as presented in Higgins and Modeer (1996), was the study conducted by Holtz and Gibbs (1951). They found that plasticity indexes ranged from 5 to 12 percent, and liquid limits from 28 to 34 percent. A minor correlation was found between plasticity index and grainsize; higher clay (<2μm) contents giving rise to higher plasticities. Total stress shear strength parameters c and ϕ were tested by the triaxial (UU) method, cohesion was found to vary considerably with water content; angle of internal friction remained constant between 30 to 34°, and cohesion was tested to be as high as 448kPa in samples of high density and high clay contents. Higgins (1987) conducted tests himself and found that in the unconsolidated undrained triaxial test cohesions ranged from 21 to 69 kPa, and the angle of internal friction from 9 to 21 degrees. Consolidated undrained tests were also conducted, with cohesions reported between 0.5 to 30 kPa and an angle of internal friction between 27 and 29°. No moisture contents were reported for either tests, or was a detailed description given of how the tests were performed.

2.3.3.1 Vicksburg Loess

Matalucci et al (1970) explores grain orientation (which dips with an average of 4° towards the west, although he does not say why this occurs as loess is typically massive) in Vicksburg Loess and the effect it has on shear strength when tested at different angles to grain orientation. Strength was maximum when σ_1 was perpendicular to grain orientation and weakenest at 45°. Results are presented for insitu samples as well as remoulded samples, the angle of internal friction drops from 34° to 31° when testing angle to grain orientation is rotated 45° from normal and samples are air dried. It ranges from 24° to 21° when the samples are at 16% water content. Cohesions drop remarkably from 44 p.s.i. (303 kPa) to 33 p.s.i (227.5kPa) for air dried loess, and 11 p.s.i. (75.8 kPa) to 6 p.s.i. (41.4kPa) for 16% water content. Atterberg limits for Vicksberg Loess have been reported as 26.5 for the liquid and 24.5 for the plastic (which calculates to a questionably low plasticity index).

Matalucci also observes an expansion in Mohr circle size when hydrostatic pressures exceed a certain threshold, and he ascribes this as natural behaviour in undisturbed specimens. It is similar to the argument presented by Kie (1988) in which he describes bimodal shear strength behaviour due to Lanzhou loess' ability to collapse.

2.3.3.2 Iowa Loess

In a study on Iowa Loess, Kane (1968) investigated shear strength properties and introduces the critical water concept, where negative pore pressures in the clay binder are measured to indicate the clay's relative saturation and indeed the overall strength behaviour of Iowa Loess. The study was undertaken to provide a report for the Iowa Transportation Board to clarify some of the observed soils practical behaviour.

Some physical properties of Iowa State Loess are:

1. grain size range of 10-30% clay (<2μm), with the rest being silt and sand (not given)

2. clay component is made up of a dominant amount of montmorillinite with a subordinate amount of illite
3. at natural water content microscopic voids (clay size) are saturated
4. silt sized particles don't touch each other, being separated by the clay fraction, hence soil behaviour is dictated by the clay fraction
5. liquid limits are 25 to 45%, and plasticity indices are 5 to 25.

Kane states that the loess structure is maintained by the strength provided by the clay binder, and that the primary cause for loss in strength is the wetting of that binder with high pore pressures causing a reduction in frictional resistance.

Of the seven sites considered for a test pit one was selected two miles north of Iowa City in an area of gently rolling hills. The loess in this area overlies sand and has a thickness of about 4.6m. The pit itself was dug to a depth of about 1.8m and was about 1.1m square in plan. Two piezometers were also sunk upslope and downslope of the test pit to measure seasonal variations in the water table. The sampling procedure consisted of the removal of loess blocks (8in310in312in), which were then wrapped in plastic, then wet rags, and again in plastic; three water content measurements were taken from each block. Upon arrival at the laboratory the blocks were subdivided again to dimensions of 3in33in36in ready to be hand carved for the triaxial test. The blocks were then wrapped in aluminium foil, dipped in molten paraffin, wrapped in a plastic bag and finally stored in a moist room so that water levels could be maintained over the entire eight month test period if needed.

Water contents were arbitrarily targeted at 8%, 14%, 20%, 26% and 32%; the in situ water content was measured to be approximately 26%. To achieve a higher water content than the in situ small samples were taken from the block samples to determine the overall water content, then the amount of water needed to bring it up to the nominated target was calculated and that amount was then added to the block sample by the following process: samples were unwrapped, sprayed with water, rewrapped and left for approximately one week to attain an overall even distribution of water over the entire sample. The process was repeated until the desired water content was reached. To achieve water contents less than the insitu value the samples were unwrapped and left to air dry, then rewrapped again to attain an overall even distribution of water. This process was also repeated until the desired water content was reached so as not to cause cracking. Overall, water contents ended up being plus or minus one per cent of the original target.

Once samples were prepared to the desired water content they were trimmed (trimming technique was not given) for triaxial testing, in a moist room, to a diameter of 1.5 in (38mm) and a height of 3.4 in (86mm). The triaxial test method used was quick-undrained/unconsolidated with no measurement of pore pressures. The tests were conducted at a strain rate of six per cent per minute. Testing was stopped when failure occurred, which was interpreted to be when axial stress decreased or 20% strain was reached (although he does not say why he stops at this figure).

Results from triaxial testing were manipulated using modified Mohr calculations or stress path data. $(\sigma_1+\sigma_2)/2$ was graphed against water content for each cell pressure, and a relation was established so that shear and normal stresses could be interpolated for a nominated water content. Those relationships were then graphed as Modified Mohr-Coulomb diagrams (modified from stress path data). Figure 2.12 summarises all of Kane's (1968) modified Mohr-Coulomb total stress failure envelopes.

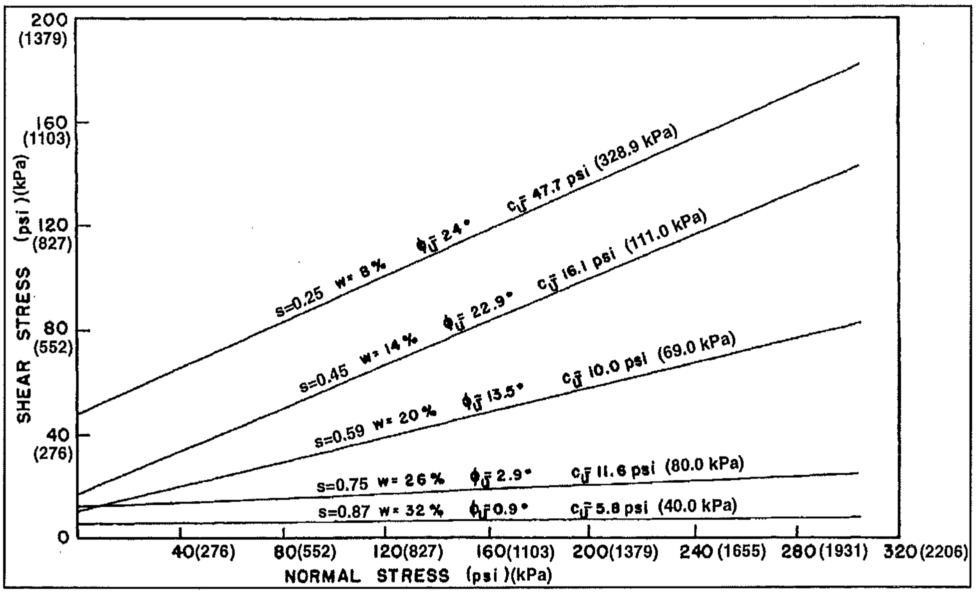


Figure 2.12, Modified Mohr-Coulomb failure envelopes for Iowa Loess (modified from Kane, 1968)

Angle of internal friction was graphed against water content (figure 2.13), and as can be seen there is a sharp drop between 14 and 24%. Kane explains that this is the result of a sharp contrast in the degree of saturation between these two water contents.

Apparent cohesion was also graphed against water content and not surprisingly the relationship is inverse (figure 2.14). A power law seems to dictate the behaviour of cohesion, but this is not stated by Kane. One explanation for the steepening of the curve is the rapidly increasing negative pore pressure, which will occur once the clay binder dries out below saturation. This is another interpretation of what Kane postulates as a critical water threshold, in which behaviours above the threshold are more or less the same but once below the threshold behaviour changes rapidly.

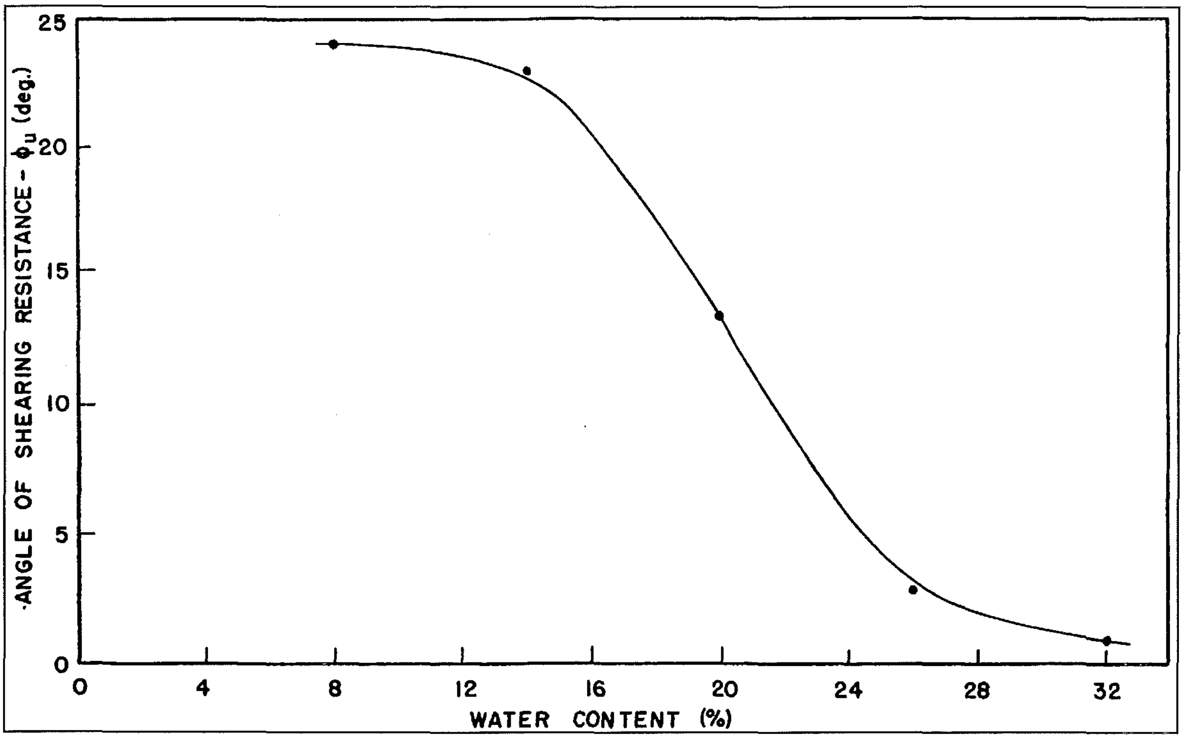


Figure 2.13 Angle of internal friction vs water content for Iowa Loess (from Kane, 1968)

Kane concludes that the rapidly increasing shear strength below the so called critical water (14-16%) content is due to capillary effects within the clay binder, and in this state the water content of the binder represents the water content of the soil as a whole. Cohesions measured above the critical water content are attributed to small capillary effects in the silt sized particles. Kane's recommendations based on natural water contents throughout the year is that soil strength measured below the critical water content can be misleading as natural water contents are normally above, therefore shear strength parameters measured above the critical water content should be used.

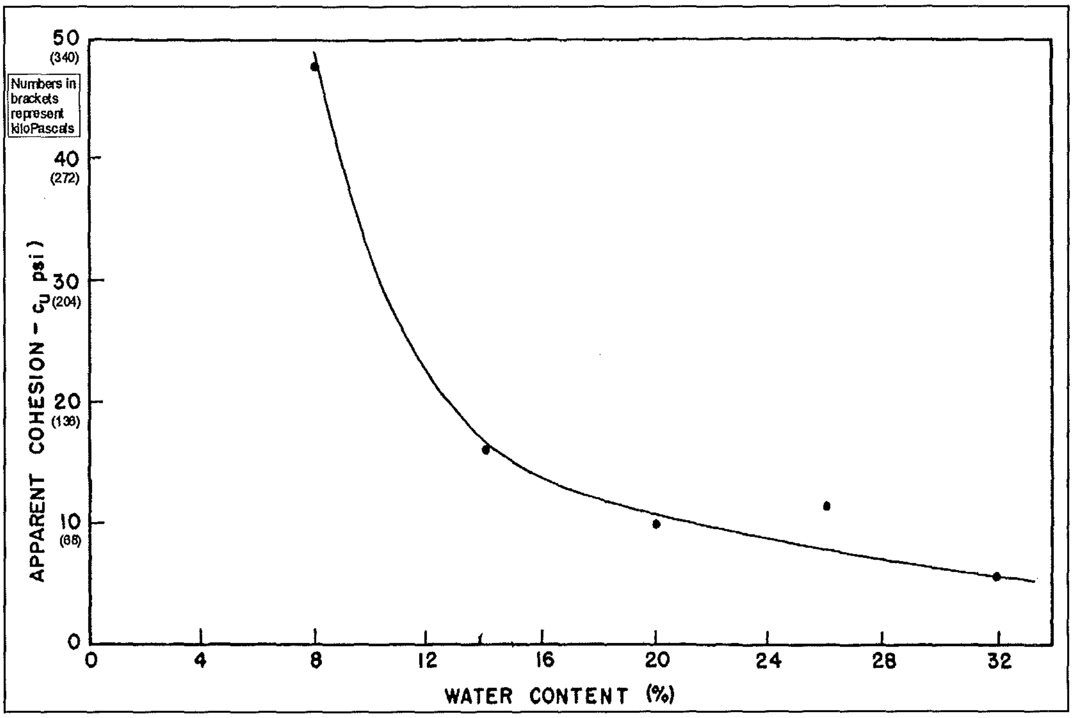


Figure 2.14 Apparent cohesion vs water content for Iowa Loess (from Kane, 1968)

2.4 Banks Peninsula Literature on loess

In all, four research studies are reviewed in this section. Most work carried out on Banks Peninsula Loess concerns slope stability and therefore shear strength is invariably reviewed, but is not necessarily the central focus. A variety of test methods have been employed to determine the shear strength parameters, and most are in agreement as to what those parameters are, roughly zero cohesion and about 30^{o} angle of internal friction.

2.4.1 Barry's Bay Loess

The first piece of research was conducted by Mackwell (1986) and centres on two "slide-avalanche-flows", those being the Wainui Dump slide and the French Farm slide, Akaroa Harbour; both slides were the result of two days of intense rainfall in August, 1975. The project itself comprised rainfall monitoring in 1984 and 1985, engineering geological mapping and model development, plus laboratory testing including undrained unconsolidated triaxial testing. The slip surface was thought to have developed along permeability barriers such as the bedrock interface and compacted clayey layers creating permeability tables.

The specimens prepared for triaxial testing were collected at natural water contents from cut faces which had been cleared of any highly weathered surficial debris. The soil tested was loess-colluvium. Tests were interpreted in sets of threes constituting the three cell pressures required for a Mohr-Coulomb failure envelope to be calculated those being 50, 100 and 200 kPa; she does not give reasons for her cell pressure selections.

The shear strength parameters for the Wainui dump slide loess-colluvium (intact slab material not failure surface material), tested at 20% water content, were calculated to be 86kPa and 32^{o}, whilst the French farm slides, tested at 16.5% water content were calculated to be 114 kPa and 30^{o}. Mackwell states that these parameters can only be used in the total stress state, and that effective stress parameters would be impossible to acquire due to the partially saturated nature of the loess soil.

Goldwater (1990) researched shallow slope stability at Allandale, Lyttleton Harbour. Unconfined compression tests, hand vane shear tests and shear box tests were used in order to determine the shear strength parameters, and in turn present a slope stability model. For the unconfined compression testing of the "P" (Goldwater uses Hughes' (1970) layering model) layer there appears to be no clear inverse relationship between water content and shear strength, however, there is an inverse relationship in the "C" layer loess. Results for the shear box tests are 20kPa and 29.2^{o} for undisturbed "P" layer loess at 19.5% water content.

2.4.2 Birdlings Flat Loess

McDowell (1989) produced a remedial engineering geological model for three residential sites on the Port Hills. Two of the three sites, Coleridge Tce and Westmorland Subdivision, were excavated in Port Hills Loess, whilst the other site concerned volcanic colluvium. McDowell states that mapping was essential to delineate the different loess types into colluvium, insitu and the identification of the pan layer. Of all the Banks Peninsula literature reviewed, McDowell provides the most comprehensive laboratory triaxial testing account and offers measured arguments for the interpretation of his results, including the type of test chosen and accompanying reasons.

Undrained unconsolidated triaxial testing was performed on both Loess types (Coleridge Tce and Westmorland), and in all about 60 tube samples were prepared for triaxial testing. Some were tested at natural water contents, whilst others had the water contents increased by applying a standing head of water directly into the stainless steel tube and subsequently placed in the fog room, hence a relation between shear strength and water content could be formulated. The graph in figure 2.15 provides a summary of shear strength parameters cohesion and angle of internal friction; the highest cohesion recorded was 178kPa for Westmorland loess at a water content of 8.5%, and the lowest equaling zero cohesion at a water content of 19%. McDowell claims that both types of loess conform to the same strength behaviour, although it is debatable that not enough samples have been tested to interpret this relation as the Westmorland samples were not tested at higher water contents. Angles of internal friction for all samples tested remain relatively constant, however, cohesion shows bimodal behaviour with a steeper relation evident in water contents between six and twelve percent. McDowell believes this is due to a greater effect of negative pore pressures (soil suctions) in this water content range.

Also of interest are McDowell's arguments for the type of test he chooses, which are mostly based on practicality. He considers that the sampling procedure for the shear box test makes it time consuming.

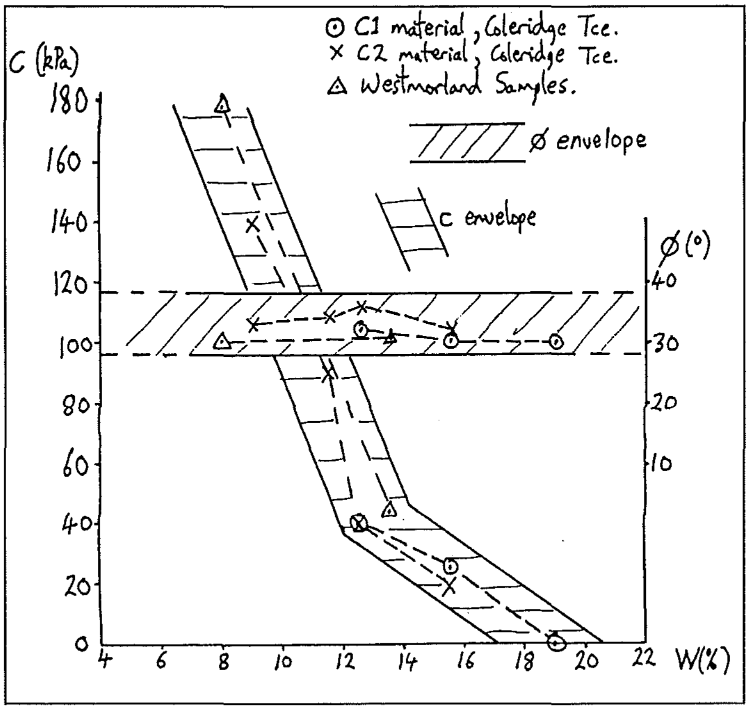


Fig 2.15 Apparent cohesion versus water content for Coleridge Tce and Westmorland Loess, Port Hills (from McDowell, 1989)

In terms of measuring pore pressures to produce effective stress parameters, McDowell argues that the partially saturated nature of loess makes this impossible, whereas the drained consolidated test would take too long to saturate the loess and does not mimic the in situ behaviour of loess, which tends to fail rapidly. Measuring total stresses appears to be the most applicable way to measure loess shear strength, as pore pressures rarely dissipate in loess' quick mode of failure behaviour.

In an honours civil engineering project, Ensor (1999) uses a drained triaxial test to measure the strength properties of Whaka Tce Loess valley fill. Results for index tests are as follows: 1) Grain size comprised clay 10%, silt 70% and sand 20%; 2) Bulk density was recorded at 1740kg/m^3; 3) void ratio was 0.67 and porosity 0.40; and 4) Liquid limit was 27%, plastic limit 14% and plasticity index of 13 (which is high for Port Hills Loess). The triaxial test itself was conducted at 0.015mm/min and saturation (which was achieved by back pressure). Testing took at least 10 hrs per sample to complete, and three samples were tested to evaluate the Mohr-Coulomb failure criteria. The samples tested were recorded to have an angle of internal friction of 34° and cohesion of 0 kPa (figure 2.16).

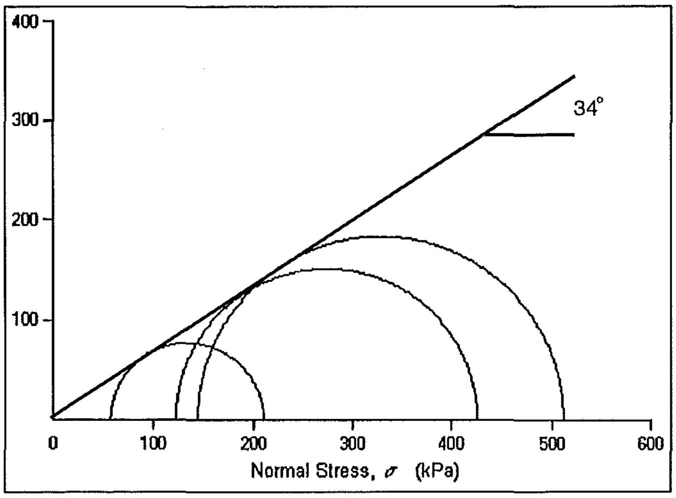


Figure 2.16 Mohr's Circle and failure envelope for the consolidated, drained triaxial testing (modified from Ensor, 1999).

2.5 Data synthesis and comparisons

In the literature that has been reviewed undrained unconsolidated triaxial shear testing is the consistent method to obtain parameters cohesion and angle of internal friction, however, there are some differences in the way the data is analysed and interpreted. The data does provide contrast and comparisons that should not be ignored. Table 2.1 summarises the data in the literature reviewed above.

Direct comparisons for shear strength parameters can be made with most of the Banks Peninsula, literature namely Mackwell (1986) and McDowell (1989) as the testing method is identical (undrained/unconsolidated triaxial). McDowell provides the best comparison of results as his testing method and manipulation of raw data is identical to that used in this project. Direct comparisons cannot be made for loess from other parts of the world because of its differing intrinsic properties. However, Kane provides the best comparison, and similar shear strength and water content behaviours can be seen even though Banks Peninsula loess does not collapse in contrast to soils from North America. He provides the most extensive laboratory description for soil sampling and preparation and a lot of his ideas were used, especially in terms of water content manipulation and analysis of results. Of note is the similarity in behaviour between Kane's and McDowell's

Researcher	Cohesion	Angle of internal friction (degrees)	Liquid and plastic limits	Grain size	Test type
Higgins & Modeer (1996) North American Loess	as high as 448kPa *Higgins(1987) 21-69	30 ~ 34 9 ~ 21	LL = 28 ~ 34 PI = 5 ~ 12	Definition: clayey loess > 21% clay sandy loess > 8% sand	Unconsolidated/undrained Triaxial
Kie (1988) Chinese Plateau Loess	3~5kPa (before collapse) 10~15kPa (after collapse)	18 ~ 22 (before collapse) 29 ~ 32 (after collapse) * 16.7 @ 8% water content > 6 @ 20%	LL = 27 ~ 30 PI = 10 ~ 14	clay = 3 ~ 4%	Consolidated/undrained Triaxial
Matalucci (1970) North American Loess (Mississippi State)	303.4kPa (air dried) 75.8 kPa (16% water content)	34 (Air dried) 24 (16% water content)	LL = 26.5 PL = 24.5	clay = 9%	Unconsolidated?/undrained Triaxial
Kane (1968) North American Loess (Iowa State)	328.9 kPa @ 8% water content 111 kPa @14% 69 kPa @ 20% 80 kPa @ 26% (soil suction effects?) 40 kPa @ 32%	24 @ 8% water content 22.9 @ 14% 13.5 @ 20% 2.9 @ 26% .9 @ 32%	LL = 25 ~ 45 PL = 5 ~ 25	clay = 10 ~ 30%	unconsolidated/undrained Triaxial
Ensor (1999) Whaka Terrace Birdlings flat Loess	0 kPa (saturated)	34 (saturated)	LL = 27 PL = 14 PI = 13	clay = 10% silt = 70% sand = 20%	Unconsolidated/undrained Triaxial
Mackwell (1986) Barry's Bay Loess-colluvium	86 kPa @ 20% water content 114 kPa @ 16.5%	32 @ 20% water content 30 @ 16.5%	LL = 35 ~ 40 PI = 15 ~ 20	clay = 16 ~ 30% silt = 60 ~ 69% sand = 9 ~ 18%	Unconsolidated/undrained Triaxial
McDowell (1989) Colleridge Terrace Fragipan Birdling's Flat Loess	140 kPa @ 9% water content 90 kPa @ 11.5% 40 kPa 12.5% 20 kPa 15.5%	33 @ 9% water content 34 @ 11.5% 36 @ 12.5% 32 @ 15.5%	LL = 21 ~ 27 PL = 16 ~ 20 PI = 5 ~ 10	clay = 14 ~ 19% silt = 69 ~ 74% sand = 11 ~ 12%	Unconsolidated/undrained Triaxial
Goldwater (1990) Barry's Bay Loess	20 kPa for "P" layer loess @ 19.5% water co 6 kPa for "C" layer loess	29.2 for "P" layer loess @ 19.5% water cont 30.2 for "C" layer loess	LL = 22 ~ 26 for "P" layer loess PL = 24 ~ 30 for "C" layer loess PI = 6 ~ 9 for "C" layer loess	clay = 14.5% for "P" layer loess silt = 76% for "P" layer loess sand = 9.5% for "P" layer loess	Shearbox

Table 2.1 Summary of loess shear s rength parameters and soil properties from both international and local research.

results (figure 2.14 and 2.15), which is seen in the steepening of the curve that happens at around 14-16% water content for Kane and 12-14% for McDowell.

It is important to note that no testing involving calculation of matric suction during triaxial testing (analysis 2.2.2.3) was found in the literature for loess. Kane, however, does measure negative pore water pressures to determine the water content within a clay binder, and subsequently determine a water content range in which strength properties vary considerably with small changes in water content, but no effective stress parameters are sought.

From table 2.1 it can be seen that soil properties for Banks Peninsula Loess are comparable to those of North America and China. Liquid limits are in the mid to high 20's, and plasticity indexes are about 10 to 20; clay contents are also similar, although the Asian Loess described has a low clay content of 3-4%. Similar variability is present within Banks Peninsular Loess with regard to soil properties.

2.6 Formulation of research project

2.6.1 Evaluation of shear strength analyses as applied to Banks Peninsula Loess

The overall aim of this thesis "is to determine the dependence that shear strength has on water content". In order to achieve this water contents have to be manipulated, resulting in soil that is dry, partly saturated or saturated. In light of the above statement it would seem most logical that the saturated soil (effective stress) and partly saturated soil (matric suction) analyses would produce the most scientifically correct result to determine the parameters cohesion and angle of internal friction (using these analyses the parameters would in fact be effective cohesion and effective angle of internal friction). However, in light of loess' behaviour and the test equipment available for the project at hand, total stress analysis has been chosen as the best way to model shear strength parameters to achieve this thesis' aims. This is because:

1. For the effective stress analysis there are two laboratory test methods to measure c' and ϕ', and these use an undrained unconsolidated triaxial test

with measurement of pore pressure or a drained triaxial test. In the first case pore pressures in loess test specimens are extremely difficult to measure; samples can take up to 6 months to saturate due to loess' very low permeability ($K<10^{-7}$). It would not be possible to carry out the volume of tests required in the short amount of time to achieve the aims of this thesis. In the second case, the drained triaxial test, tests are conducted at such a slow rate that it is thought that the slow rate of compression would not effectively model failure associated with Banks Peninsula loess as drainage does not occur in failure (i.e. development of high pore pressures).

2. In the matric suction analysis the triaxial test equipment is simply not available to evaluate the matric suction ($(u_a-u_w)\tan\phi^b$). Testing requires special porous discs, which can measure minute changes in air pressure and it is they that are not available.

The total stress analysis accompanying the undrained unconsolidated triaxial test method would appear to be the best choice, based on practical and modeling considerations. Again, in light of this thesis' aims, varying water content becomes an important factor in the test method used. Triaxial test specimens, being smaller and contained in sealed stainless steel tubes, appear to be the more practical choice over the larger shear box specimens, which would pose difficulties in manipulating water contents because specimens cannot be sealed and stored as effectively.

It was therefore decided that the undrained unconsolidated triaxial test without measurement of pore pressure and with total stress analysis would be used to calculate shear strengths of Banks Peninsula Loess.

2.6.2 Evaluation of loess layer sampling

The layer model that was provided by Hughes (1970) in section 1.4.2 is also useful for separating Banks Peninsula Loess in terms of its geotechnical properties and this is also true for measured shear strengths. However, because there are three different layers there will be three separate shear strength test results for Banks

Peninsula Loess. It was decided due to time constraints that only one loess layer be sampled.

The "S" layer is a good candidate for sampling as it is shallow, providing easy access for sampling, and is prone to strength losses and slope failure in the form of turfmat slides (Bell and Trangmar, 1987). The "C" layer generally has a low permeability and often acts as a shallow aquitard, therefore strength losses due to water infiltration are not an issue with this layer; it is also not always present making site selection difficult. The "P" layer is also a good candidate for sampling, as it is also prone to erosional problems and strength losses, and it is the deepest/thickest layer therefore having the greatest influence in any site works (most foundations go down to the "P" layer). As the "S" can easily be removed out of any site works it was thought that the "P" layer would be the most appropriate for sampling and shear strength analysis.

2.6.3 Evaluation of sample depth, subsequent triaxial cell pressures and testing rate

Recommendations given by Kevin McManus (Senior Lecturer in Geomechanics, Civil Engineering, University of Canterbury) suggested the use of only three triaxial cell pressures (instead of more) to reduce scatter and increase repeatability of test results. The cell pressures he recommended based on average soil depths were 50kPa, 100kPa and 150kPa, this represents soil depths of approximately 3m, 6m and 9m, which corresponds with the depths associated with "P" layer occurrence. However, cost and time constrained the depth to which test pits could be dug (3metres). Testing rate was recommended by Dr McManus at 1.5mm/minute, which is quite high but appropriately models failure in loess which is thought to occur quickly under heavy rainfall conditions and subsequently high pore pressures.

2.7 Synthesis

1. International literature on loess shear strength is sparse and not consistent, especially in regards to the analysis of triaxial test data. However, there are

a lot of similarities in soil properties, and shear strength for all loess types decreases with increasing water contents. Kane (1968) provides the best comparisons and discussion. His is the only project that has the prime aim of evaluating strength variability with water content.

2. Banks Peninsula Loess shear strength analysis literature is somewhat more consistent than international literature, with most studies using the undrained unconsolidated triaxial test method and total stress analysis. McDowell (1988) provides the best study for comparison, and his have similarities to results presented by Kane (1968).

3. Synthesizing the review of basic shear strength techniques and loess literature, it was decided that the total stress analysis was the best method to interpret triaxial test data, and that the undrained unconsolidated triaxial test was the most appropriate laboratory testing technique because it models undrained failure (rapid failure due to high rainfall) and samples can be manipulated easily to vary water content.

4. The "P" layer was evaluated to be the most appropriate layer to sample because it was the easiest to sample and has the greater effect on most site works (most foundations and services are placed in this layer).

5. Cell pressures for triaxial testing were conducted at 50kPa, 100kPa and 150kPa, which correspond to a maximum depth of 9 metres. Sampling was conducted at a maximum of 3 metres depth. Triaxial testing rate was 1.5mm/minute using a Wyckem Farrance, Tritech 10 triaxial shear testing machine.

CHAPTER 3

Test sites and methodology.

3.1 Introduction

This chapter details field locations, test pits and soil sample descriptions for the four test sites, they are: 1) Moncks Spur; 2) Stonehaven Subdivision; 3) Worsley's Spur; 4) Duvauchelle. It explores and develops the method used to extract in situ samples for the dry loess. Site selection criteria are discussed in section 1.5.3, which included variation in geography, cost, and for the Whaka Terrace site availability of fill. Strength data for each of the four sites are presented and discussed in Chapter 4, Whaka Terrace field site is discussed in detail in Chapter 5.

3.2 Field sampling procedures

For the most part of the year loess is a very hard and relatively dry soil, and therefore difficulties arise in the sampling of loess as the material is somewhat brittle. The time of sampling for the first test site was in the dry months of November and December (i.e. early to mid summer), when rainfalls are low, and hot dry north west winds buffet Banks Peninsula. At this time, it was thought cost effective to use existing site works (services trenches and road cuts) for the purposes of soil sampling, however, as will be seen below this would become impractical. As the Moncks Spur field site was the first sampled, it became the site for developing a consistent and effective test method.

Fifty tubes approximately 25cm long were made for sampling from 35mm diameter stainless steel tubing so that sampling time could be reduced by handling the large volume of samples that would need to be taken for immersion (explained below). 35mm diameter tubing was used instead of the required 38mm tubing as it was all that was available in N.Z. (only 3 stainless steel 38mm tubes were available for sampling from the University of Canterbury's Geomechanics Laboratory). It was advised by Dr Kevin McManus (University of Canterbury) that a smaller sample diameter would not affect overall test results. Field sampling experiments involved the following as a suitable technique was developed:

1. The extraction of 35mm stainless tube samples was attempted from a services trench, which had been excavated for a stormwater drain. This was a difficult task as the soil was just too hard and dry for the stainless steel tubes to penetrate and they had to be hammered into the ground for all sites using a specialised percussion attachment. Moreover, upon extrusion of the soil from the tube samplers the soil cracked to such an extent, that they were useless for triaxial testing. Soil water content at this time was around 3-6%, which is a typical of the air-dried range for loess water contents.
2. Stainless steel tubes were also pushed into the hard loess of the services trench (described above) using a backhoe bucket, as it was thought that the percussion attachment would cause too much sample disturbance. A guiding tube was designed and made so that the sample tube would penetrate the soil at right angles. However, this technique proved unsuccessful as even the guiding tube, anchored to the soil with 225mm nails, could not achieve straight penetration and tubes bent in the process.
3. Tube samples were taken from a freshly dug road cut, where it was anticipated that the soil exposed would be higher in water content and so pose less of a problem regarding brittleness. The tube samplers required far less effort to penetrate the soil and it was hoped that extruding the soil would be successful. The extruded soil was fairly intact, but still had at least one or two breakages: the soil water content was about 10%.
4. Small pits (termed "soak pits") were then hand dug into the road cut, filled with water, covered, and left to soak for as long as it took for the water to disappear and partially saturate the surrounding soil, which was normally 1.5 days. This time sampling proved successful, with intact samples extruded in the laboratory. Water contents for this method were measured between 16 and 20%. "Soak pits" soaking soil for two to three days in the middle of site works were not exactly convenient for the contractor, so another extraction site had to be found.

5. It was then decided to machine dig a large test pit (Moncks Spur test pit, figure 3.2) separate from general site works, and solely for the purposes of study and sampling. The test pit was subsequently located on lot seven of Augusta Estates Subdivision (fig. 3.1). Small soak pits were then hand dug on the floor of the test pit stainless steel tubes were driven perpendicular (approximately perpendicular to horizontal level) to the "soak pit" floor after dissipation of contained water. Field water contents measured from this method varied from 16% - 20%.
6. Subsequent sites (Stonehaven Subdivision, Worsleys Spur, Whaka Terrace and Duvauchelle) were also sampled using a machine dug test pit. For all sites apart from Whaka Terrace, four separate "soak pits" were hand dug so that 12 samples could be taken from each, representing four groups of samples that would be prepared at a different nominated water content.

Method 1: Air dried water content	3-6%
Method 3: Natural water content collected from the bottom of a freshly dug road cut	~10%
Method 4: Modified water content (addition)	16-20%

Table 3.1 Summary of water contents collected in the development of a field sampling method.

3.3 Moncks Spur – site description

3.3.1 Location and description

Moncks Spur (fig. 3.1) was the first site to be examined as part of the Banks Peninsula Loess strength-testing program. The nature of the research required an area of loess that was exposed or able to be dug into so that samples for strength testing could be taken. Talks with David Bell (University of Canterbury) at the end of 2000 revealed Moncks Spur as a possibility for this type of research into Banks Peninsula Loess, with "Augusta Estates" Subdivision (then currently under construction) as the prime target. The site owner was subsequently contacted and field work commenced.

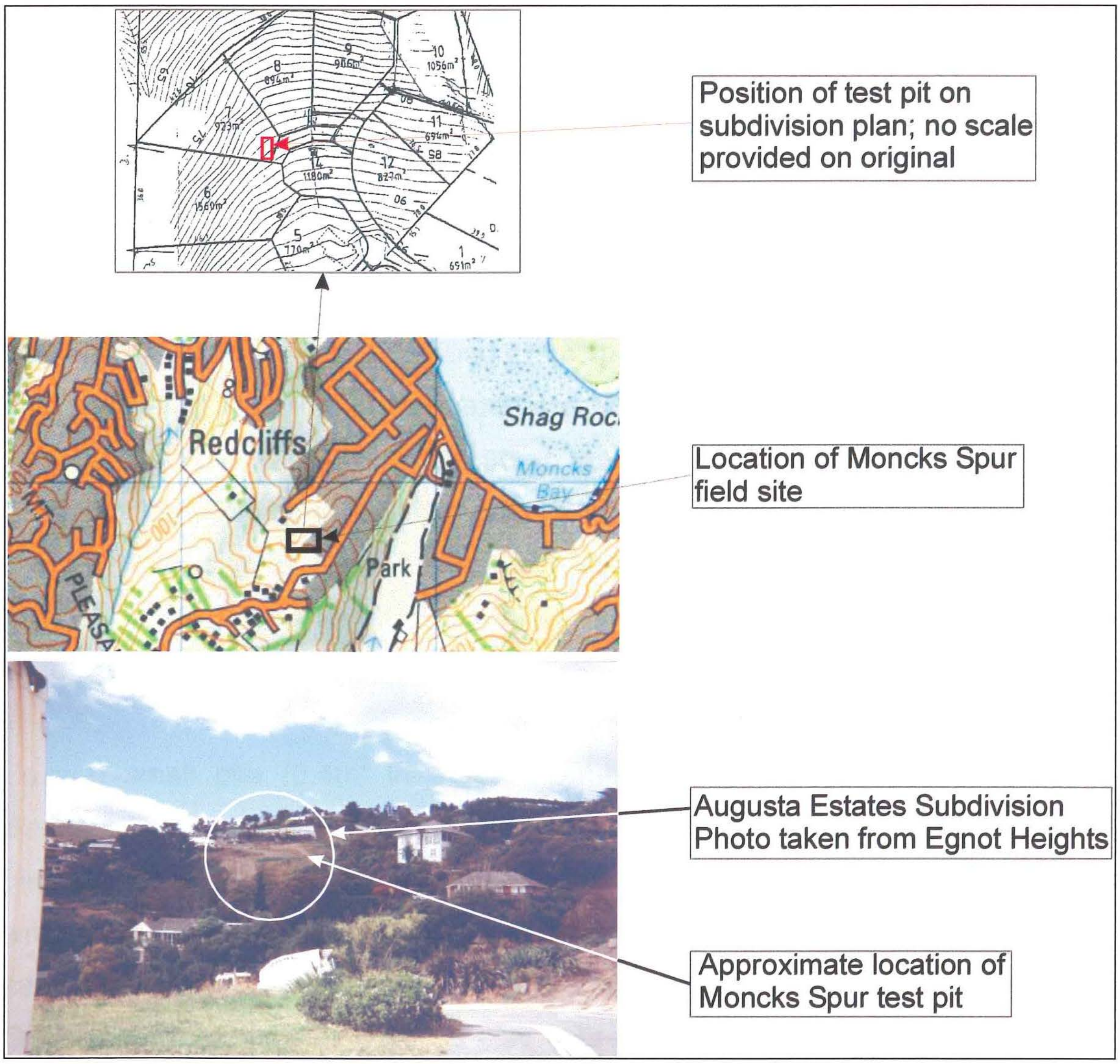


Figure 3.1 Locality map for Moncks Spur field site.

No major slumping/sliding or erosion is apparent on the proposed lots; however, there is some evidence of movement in the form of tension cracks in the gully below lot six (figure 3.1). It was interesting to note that on the road cut neighbouring the test pit on lot seven a small tunnel feature, approximately 30 cm in diameter, had appeared overnight during a storm, indicating the relatively unstable/erodible nature of exposed loess. In his subdivision feasibility report David Bell stated that there were "minor areas of fill to about two metres in depth", although none were present around the area of research and may have already been excavated.

3.3.2 Moncks Spur Test Pit

The test pit was located on lot seven of Augusta Estates subdivision for the purposes of sampling, and to allow logging characterisation of natural soil present on the spur. Test pit dimensions and photographic evidence are given in figure 3.2. The test pit was machine excavated to a depth of about 1.5 metres, although digging was difficult because of the dryness of the soil. A water content measurement was taken at the bottom of the test pit and found to be 10.1%.

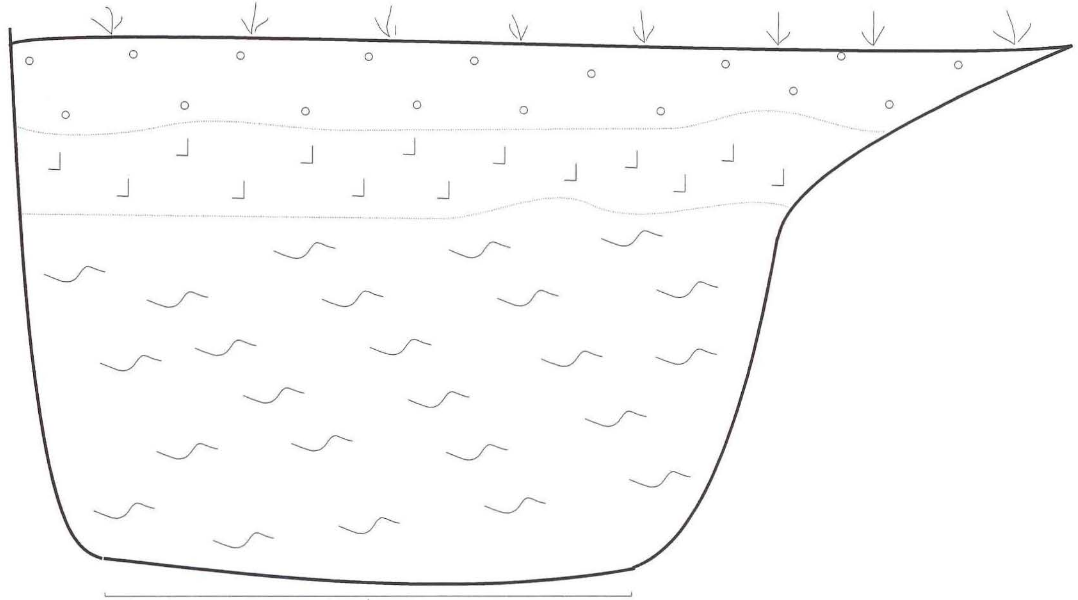
The soil profile is a typical "Birdlings Flat" profile comprising an S-layer, C-layer (fragipan), and a parent layer material or P-layer using Hughes' (1970) Banks Peninsula Loess classification system (section1.4.2). Under the Unified Soil Classification Scheme the P-layer soil, which was characterised as a sandy silt of low plasticity, symbol ML. The fragipan layer, although not obvious due to the dryness of the soil, showed typical blockiness and slight gleying. Some mottles were present, but no gammate veining.

Several small pits (0.4m^2 in area) were hand-dug and prepared/sampled as described in section 2.4. Samples were transported back to the lab in resealable plastic bags and placed in airtight plastic containers, which were lined with bubble wrap for cushioning. A total of 58 tube samples were collected, as well as bulk soil samples needed for index testing (Atterberg limits, grain size analysis etc). Grainsize calculations revealed a clay content of 10%, 76% silt and 14% sand. Results for Atterberg tests were 20 for the plastic, 26.1 for the liquid and 6.1 for plasticity index.

Legend

	Hughes (1970) classification	USCS description
	s-layer	Moderately weathered moist soft dark grey massive fine sandy silt. **OL**.
	c-layer	Moderately weathered dry lt grey hard to very hard blocky/jointed slightly gleyed sandy silt. **ML**.
	p-layer	A slightly weathered moist stiff olive grey massive silt with some fine sand. **ML.**

Soil layer boundary



Photograph of test pit face with fragipan layer outlined.

Figure 3.2 Moncks Spur test pit.	Drawn by T.J. Hughes 4/4/2002	scale 1:20	0 0.5 1.0m X = Y

3.4 Stonehaven Subdivision

3.4.1 Site Description

A new subdivision construction located near the Hillsborough Spur Reservoir was the second site for a loess shear strength-testing program (fig 3.3). The consulting engineer in charge, Nick Traylen of Geotech Consulting, was contacted and plans were made for another test-pit to be machine dug and sampled. The site is located next to an old brick works on a small gentle ridge. Loess is present as a valley fill loess-colluvium washed down from the slopes either side and at the head of the valley. Near vertical (>808) cut slopes of loess are present in the valley, and are remnants of the quarrying which was once done for brick making. They are indicated as scarps on the locality map (fig 3.3).

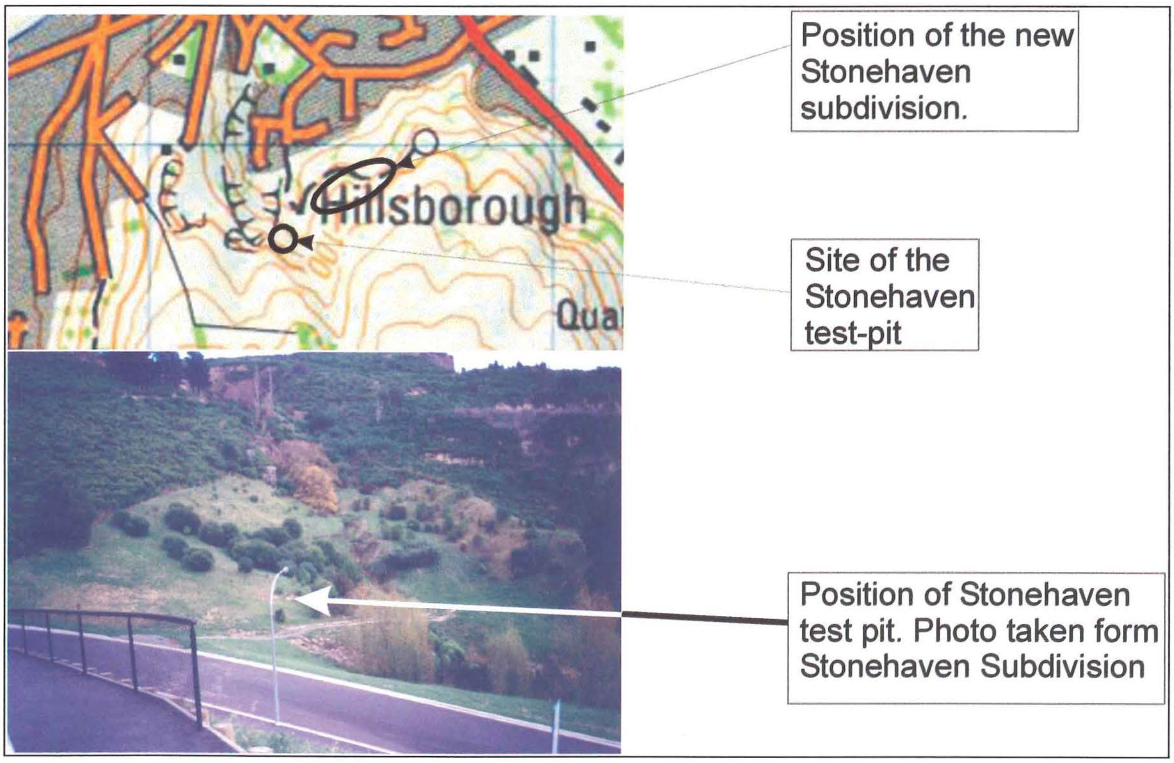


Fig. 3.3 Location of the Stonehaven Subdivision field site

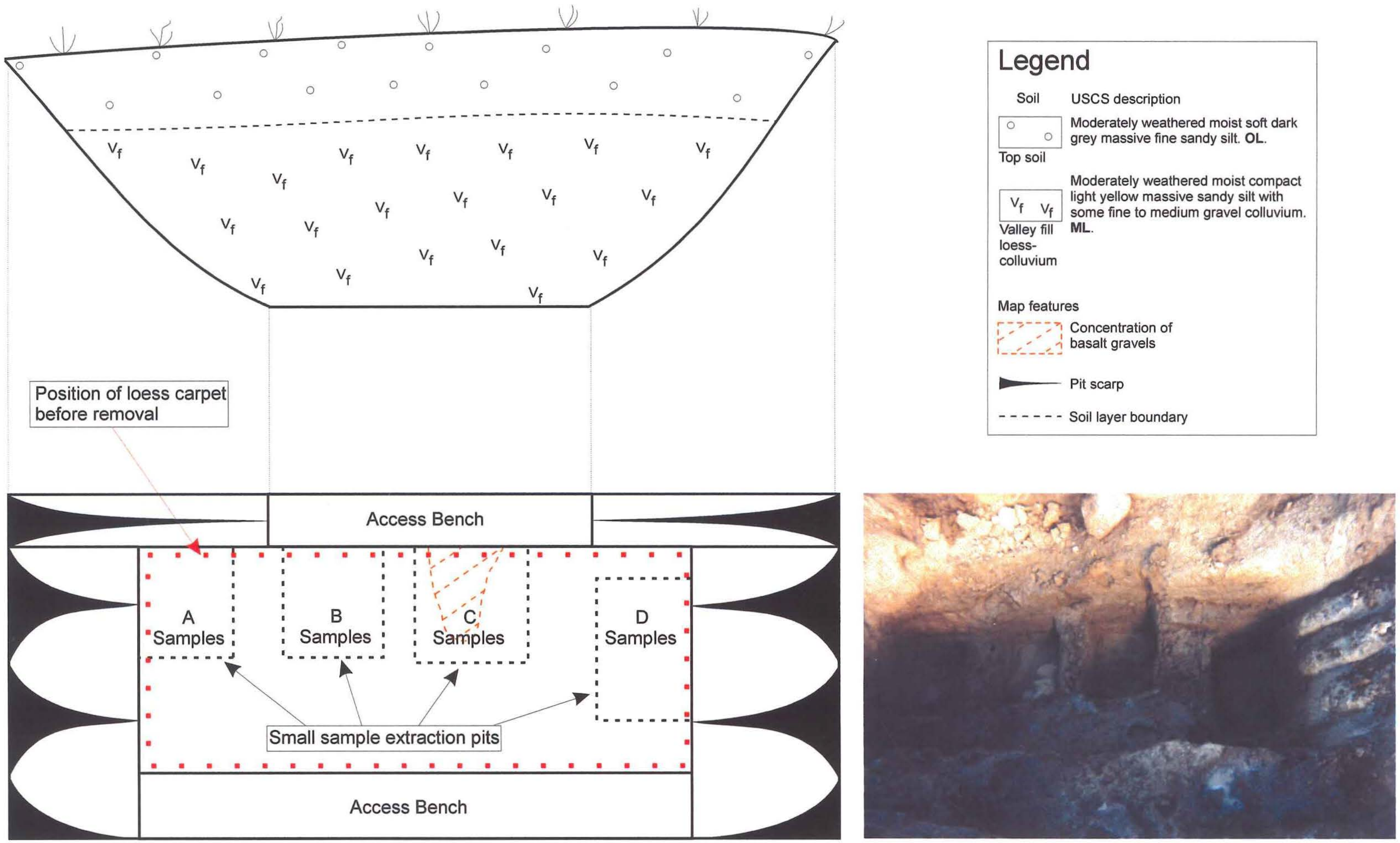
3.4.2 Stonehaven Subdivision Test Pit and Field Sampling

A test pit (fig 3.4) was machine excavated adjacent to the subdivision on Council land with the help of the subdivision contractors, Fulton Hogan Ltd. Although sites containing in-situ loess were present they were not available for excavation, and it was thought that the sampling of valley fill loess (loess-colluvium) would make an interesting comparison with primary airfall loess deposits sampled at Moncks Spur, Worsleys Spur and Duvauchelle.

The test pit was about 2.5 m long, 1.5 m wide, and 1m deep, and was dug into what was described by the contractors as "natural ground". Certainly there were no signs that the ground was composed of manmade fill. Upon excavation the test pit was logged, and full descriptions are given in figure 3.4. The soil was indicative of a valley fill/loess colluvium deposit as there was no loess layering present in the form of a fragipan, and there were small concentrations of basalt fragments; also the position of the deposit on the lower valley slopes would suggest a zone of loess-colluvial deposition. Under the Unified Soils Classification System the valley fill loess-colluvium was defined as a sandy silty with low plasticity, symbol MH.

After the "soak pits" had been hand dug and water infiltrated to the underlying soil a total of 48 samples were collected, twelve from each small pit. The colluvium did contain fine to medium basalt gravels (2-4mm in diameter), but for the most part this did not affect the sampling. However, a concentration (more than one fragment per 35mm^2 of surface area, i.e. the area that a tube would sample) of basalt gravels in the small pit which the C samples were to be removed did make sampling difficult, and samples had to be taken from around the concentration, which meant extra excavation for that particular soak pit.

Grainsize analysis results measured from four triaxial test specimens for Stonehaven Loess are: clay 5%; silt 78%; and sand 17%. One sample contained no clay at all with a higher level of sand (26.8%), which suggests that that particular sample was taken from a sandy loess horizon in the parent material. Atterberg limits for Stonehaven Loess were 26.4 for the liquid and 20.3 for the plastic, and the plasticity index was 6.1.



Plan view of test pit

Photograph showing small pits for sample extraction on the floor of main test pit

Figure 3.4 Stonehaven Subdivision test-pit	
Drawn by T.J. Hughes 20/5/02	X=Y 0 0.5m Scale = 1: 20

3.5 Worsleys Spur

3.5.1 Site description

Due to the nature of the project it was decided to seek another site that was geographically opposite on the north-facing Port Hills to the east facing Moncks Spur field site. Comparisons could be made and differences highlighted (if any) between the two sites and so, with geography in mind, advice was sought from Marton Sinclair of Eliot Sinclair Ltd about the availability of such a site. Worsleys Spur was mentioned as a possibility and became an eventuality. Figure 3.5 provides the exact locality of the spur and test pit.

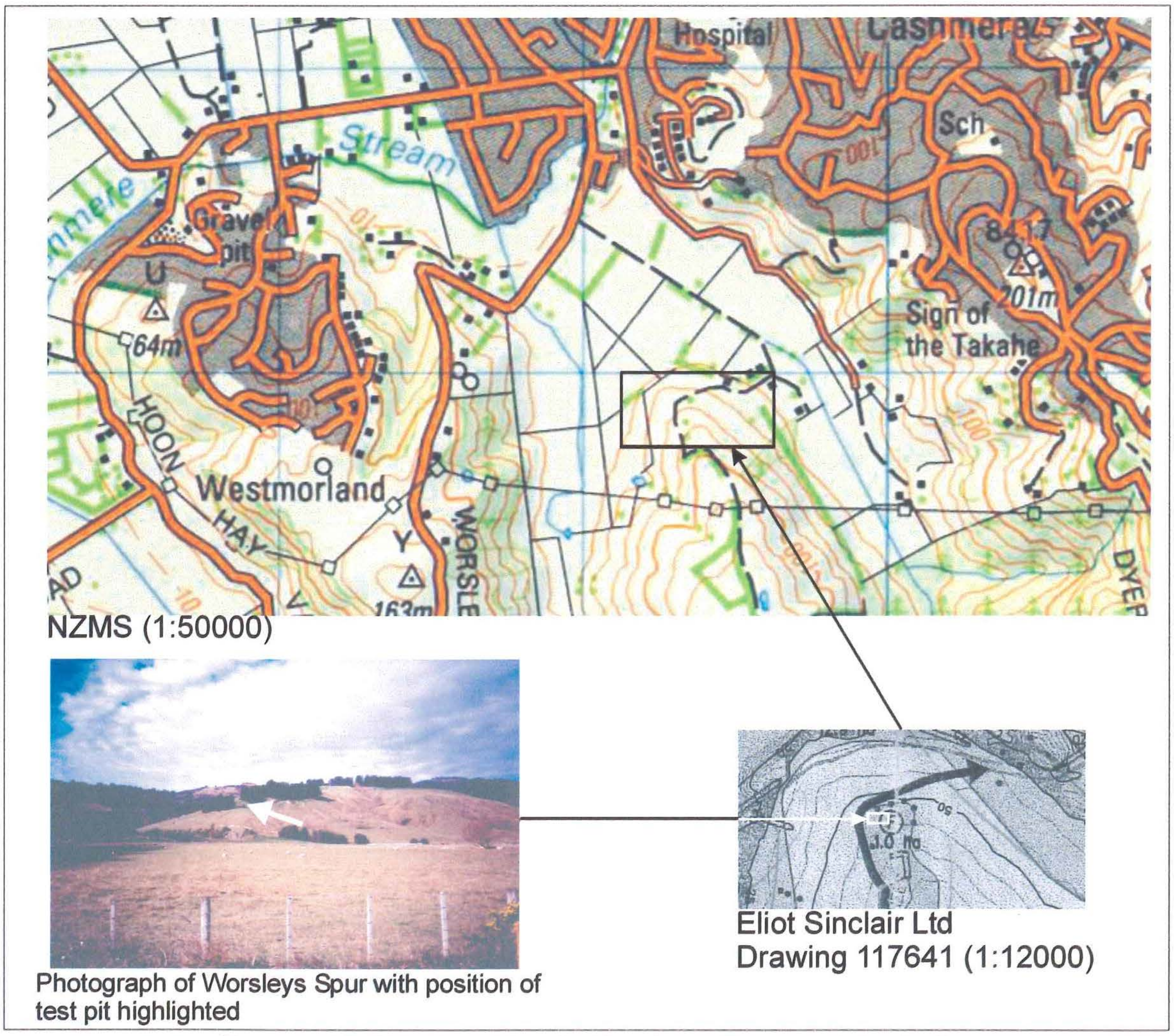


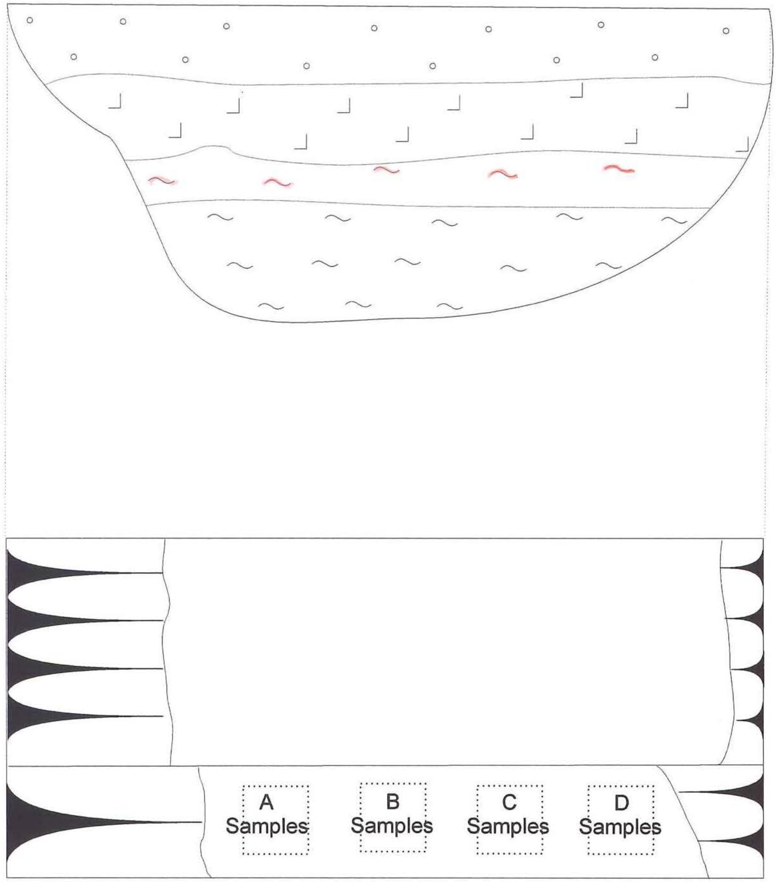
Figure 3.5 Site locality of Worsleys Spur and test pit.

The site itself is nestled in a pine-shielded paddock approximately half way to the top of the spur and it is characterized by a relatively flat bench area (sloping 5-10°), 30350m^2 in dimension. Proposed subdivision works are still in the planning phase, however it is thought that the site will subdivide into 6-7 house lots. Adjacent to the spur is the Hoon Hay Valley, which is a topographic low and suffers occasional flooding in the winter months. Tunnel-gullies line the flanks of the spur, so care was taken in choosing an appropriate site for the test pit so as not to intersect a gully.

3.5.2 Test pit and sampling program.

A test pit was machine excavated on the site described above, the pit was approximately 5m long, 2m wide and 2m deep. Typical Birdlings Flat Loess layering was exposed in the pit, and layers could be subdivided and described using Hughes' (1970) classification (refer to Figure 3.6 for face log and soil descriptions) with "S-layer" loess, "C-layer" loess and "P-layer" loess all present. USCS description of "P-Layer" loess is a sandy silt of low plasticity, symbol ML. In situ soil water content was measured at 10.5%.

After the pit was dug a separate bench was excavated to house the small pits for soil saturation and sampling (shown in figure 3.6), and four small pits were then hand dug to provide samples for four separate water content targets. 42 stainless steel 35mm diameter tubes were then extracted from the small pits after field preparations outlined in section 3.2 had been carried out. A block of soil measuring 0.4m30.2m30.4m was also extracted for shear box testing, however, the results from testing were lost in a computer hard drive failure. Hand vane shear testing was also conducted on the artificially wet loess, but this proved unsuccessful as the vanes could not penetrate the soil even though the artificially increased water contents were approximately 18%. Sand, silt and clay contents were 26%, 62% and12% respectively. Atterberg limits were 23 for the liquid, 19 for the plastic and 4 for the plasticity index.



Legend

Soil symbol	Hughes (1970) classification	USCS description
	S-layer:	Moderately weathered moist soft dark grey massive fine sandy silt. **OL.**
	C-layer:	Highly weathered moist hard olive (with mottles) blocky silt with some fine sand. **ML.**
	P-layer:	Slightly weathered moist stiff olive brown (with-grey veining) massive silt with some fine sand. **ML.**
	P-layer:	Slightly weathered moist stiff olive brown massive sandy silt. **ML.**

Map features

Soil layer boundary

Pit scarp

A Samples — Small soak pits for sample extraction

Photograph of pit wall, fragipan outlined

Photograph of Worsleys Spur test pit; small pits for sample extraction hand-dug on an elevated bench for convenience

X=Y
0 1m
Scale = 1:50

Figure 3.6 Worsleys Spur Test pit.

Drawn by
T.Hughes
2/11/2002

3.6 Duvauchelle Loess

3.6.1 Site description

In contrast to the Port Hills sites that have been sampled, Duvauchelle Loess provides results for the much more clayey Barry's Bay sub-type. Barry's Bay Loess, first distinguished from Birdlings Flat Loess by Griffiths (1973), occupies the inner harbours/harbour heads of Banks Peninsular. Barry's Bay Loess (or specifically loess from the Akaroa Harbour) displays more complex behaviour than Birdlings Flat Loess; failures are larger and more widespread, and rainfall is greater and so are clay contents. Several large landslides and instability problems have been documented in the literature, some of which have already been mentioned in this thesis (see Chapter 2, section 2.4.1 Barrys Bay loess).

In light of the project's costs and the distance to Akaroa Harbour, it was decided to seek out existing site works that could be sampled cheaply and with ease. Consultation with Marton Sinclair, consulting engineer of Eliot Sinclair Ltd, Christchurch, revealed a most suitable site in a subdivision just east Duvauchelle; no sites were available in the Gebbies Pass-Lyttleton Harbour area, which would have been more preferable in light of cost of travel. Subdivision works commenced at Ngaio Point Estates, Duvauchelle, late 2000 by Laing Construction Ltd. The site itself is a small peninsula, which subdivides Duvauchelle Bay and Robinsons Bay on the north-eastern head of Akaroa Harbour; (Figure 3.7). Entrance to the new subdivision is opposite to the turn off to Okains Bay Road, on the Christchurch-Akaroa highway.

Undisturbed primary airfall loess drapes the entire peninsula. At the north end of the subdivision (Figure 3.7) a large landslide was discovered; the failure surface was found to be an ash-fall horizon, and subsequently the unstable material had to be removed and the underlying material drained. In the eastern part of the subdivision slumping was present on a south easterly facing slope.

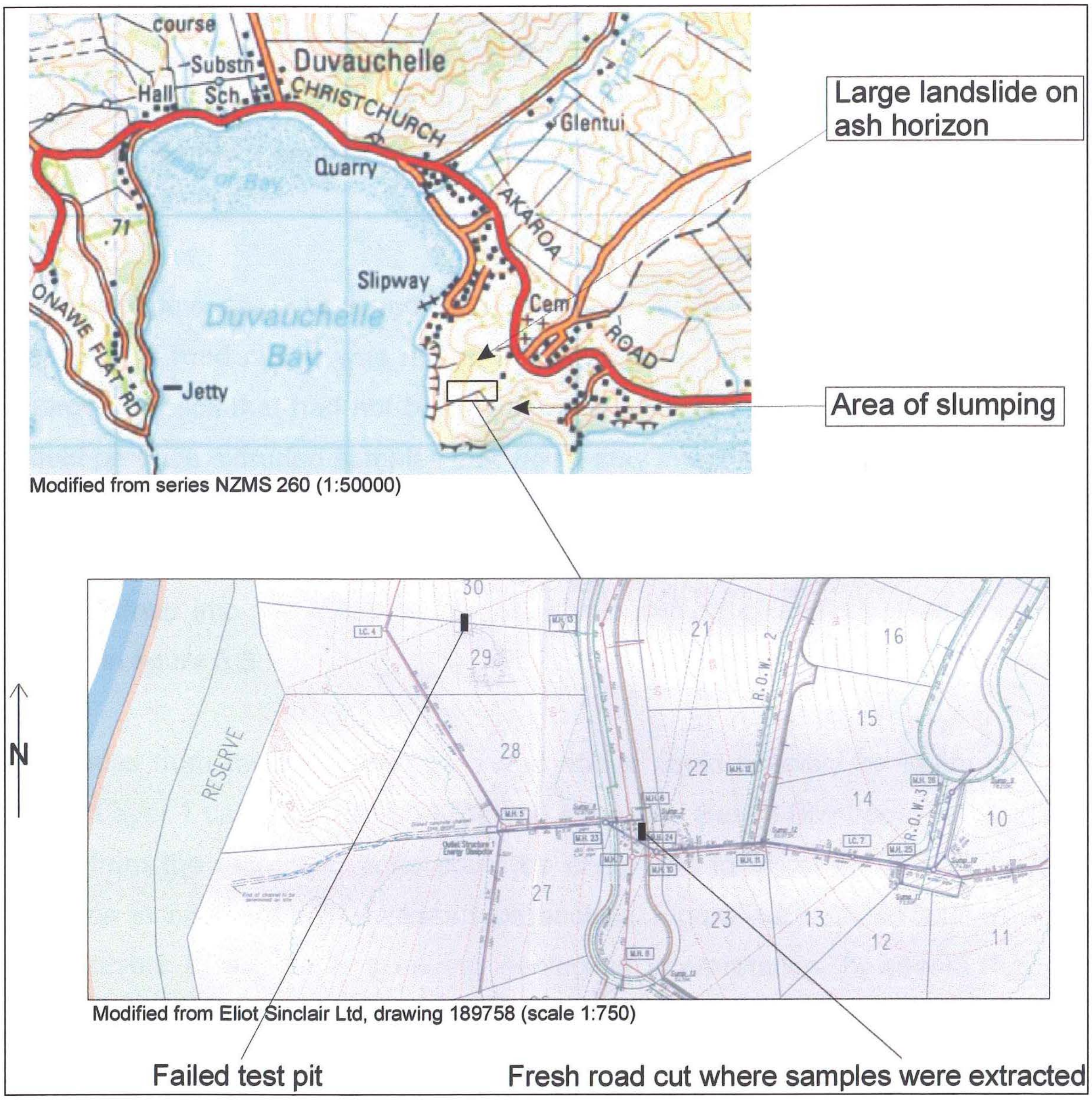


Figure 3.7 locality sketch map of Duvauchelle sample sites.

3.6.2 Test pit and sampling programme.

A test pit, 2m34m in plan, was dug to a depth of 2m on the western side of the peninsula (pit location indicated in figure 3.7). Four small pits were then subsequently hand dug in the base of the test pit so that water could be poured into them for water content elevation. Care had to be taken in positioning the small pits as there was considerable cracking and gammate veining in the base of the test pit, which would pose problems for the stainless steel tube samplers and the wetting-up process. Water was poured into the small pits and left to drain into the underlying soil; after one week the water was still present to the level at which it was poured. Ostensibly, the loess was more impermeable than Port Hills loess and the small pits had to be abandoned and sampling continued elsewhere as there was a limited amount of time; repreparing the test pit would have taken too long.

An appropriate site was found (location shown in Figure 3.7) for sampling in a freshly dug road cut. It was not known whether samples collected from a drier source, (i.e. soil that had not been artificially saturated as in the other field sites) would produce extruded samples that would stay intact for triaxial testing, however sampling from this location and method proved successful. 48 samples were extracted from the road cut in what was interpreted to be Loess Layer 4, four metres deep into the loess cover, and the positions of sample extractions are shown in figure 3.8.

The loess from the extraction zone and above could possibly be interpreted as Loess Layer 1 using Griffiths' (1973) type section of Barrys Bay Loess; specifically loess from the extraction zone could be compared to either Griffiths' C2 or C3 horizons in his Barrys Bay Loess type section. Figure 4.2 outlines four different loess layers based on engineering geological interpretation, however, Hughes' (1970) classification would suggest that the first layer is the "S-layer" loess and that the rest are "P-layer" loess. No fragipan or "C-layer" loess is present. Loess Layer 2 shows typical net-gammate veining (as described by Griffiths in his Barrys Bay type section) and mottling. Layer 3 shows deep vertical gammate veins which have been leached out to a depth of 5-10cm since the road was cut. Some cracks aren't filled with vein material and could pose drainage and piping problems. The veins themselves tend to infill shrinkage cracks, which form distinct polygonal structures. When the veins are exposed at the surface in a horizontal cut they protrude in five sided polygons, preferentially resisting erosion as compared to the parent loess in between them. Loess layer 4 (the sampled layer) was identical to Loess Layer 3, but gammate veining was not present. In situ water content was measured to be about 16%. Under the USCS description of soils Loess Layer 4 was classified as a clayey silt, symbol ML.

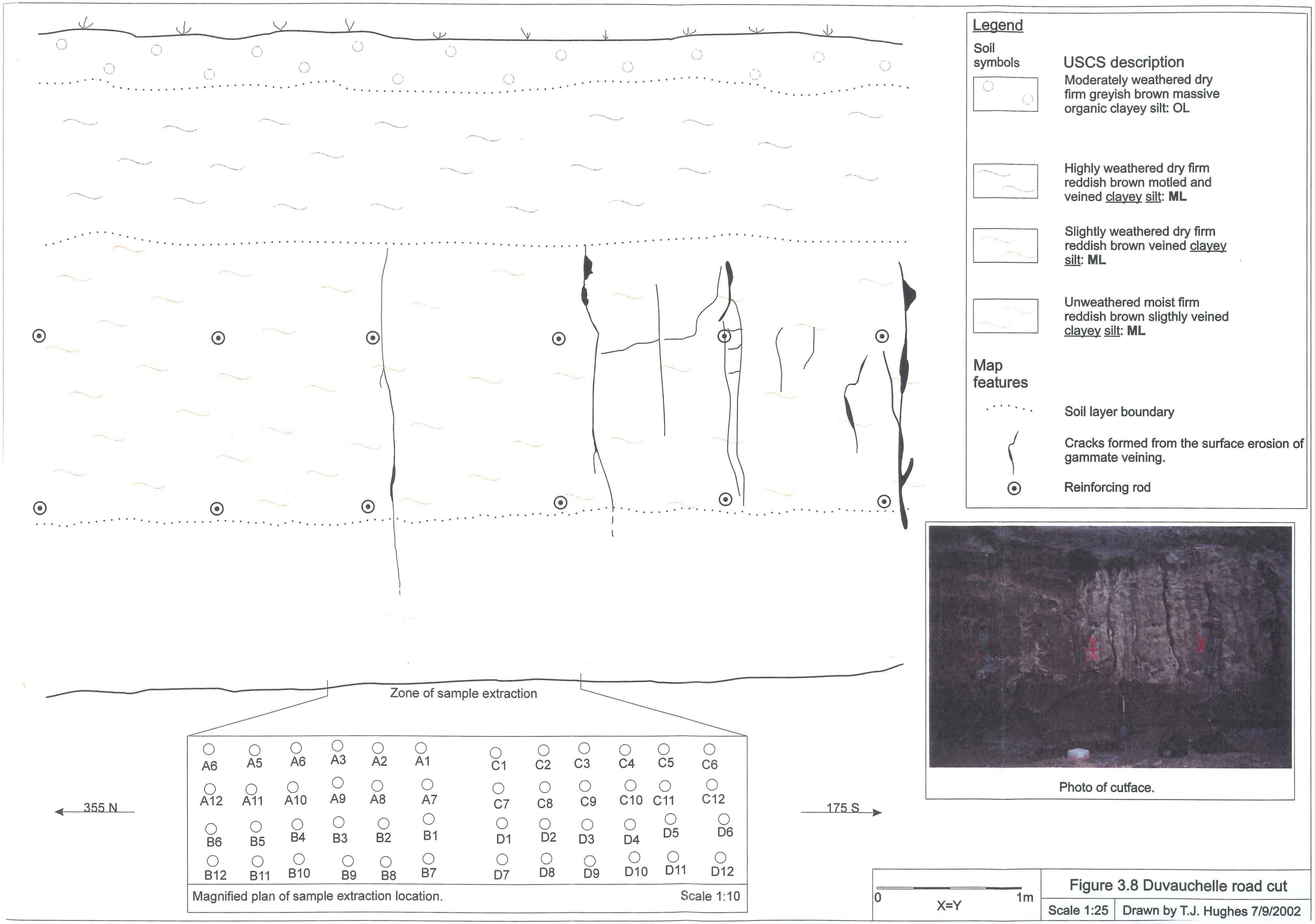


Figure 3.8 Duvauchelle road cut

Grainsize analysis for Loess Layer 4 revealed 15% sand, 65% silt and 20% clay. Atterberg limits were measured at 25.3 for the liquid, 16.5 for the plastic, and a plasticity index of 8.8%.

3.7 Soil property comparisons

Grainsize distribution for all sites tested on Banks Peninsula Loess (Table 3.2) show most soils to be either sandy silts or silts with some fine sand, Duvauchelle being the exception with slightly higher clay content and plasticity. Worsleys Spur has the highest sand content, but the lowest plasticity index. All are classified as a low plasticity silt by the Unified Soils Classification System. Grading curves for all sites are shown in Appendix 5.

Plasticity indexes confirm the USCS description as all have been measured to be quite low, ranging from 4 to 9, with Duvauchelle having a plasticity index of 9. Atterberg limit tests were conducted in accordance with NZS 4402.

Field site	Particle size distribution			Atterberg Limits			Physical properties		
	Sand	Silt	Clay	PL	LL	PI	γd Dry density	n Porosity	e Void ratio
Moncks Spur Loess In situ "P-Layer" USCS: silt with some fine sand, symbol **ML**	14.19	76.08	9.73	26.10	20.02	6.08	1.68	0.42	0.59
Stonehaven Subdivision Loess Valley fill loess-colluvium USCS: sandy silt with some fine to medium gravels, symbol **ML**	17.36	77.58	5.20	26.38	20.31	6.07	1.53	0.42	0.74
Worsleys Spur Loess In situ :P-Layer" USCS: sandy silt, symbol **ML**	25.64	62.05	12.32	22.94	18.79	4.11	1.56	0.42	0.71
Duvauchelle Loess In situ "P-Layer" USCS: clayey silt, symbol **ML**	15.11	65.41	19.47	25.25	16.5	8.79	1.70	0.36	0.56

Table 3.2 Soil properties for all 4 field sites

Physical properties were calculated from the triaxial test specimens in calculating the degree of saturation, which has been described in section 4.3.1; equations for void ratio and porosity are presented in Appendix 1 along with accompanying results for all triaxial test specimens. All have quite similar porosities at 0.42 with Duvauchelle again being the exception at 0.36. Densities range from 1.56 to 1.70 t/m^3. Generally speaking it can be concluded that the lower the density the higher the void ratio.

3.8 Synthesis

1. Four field sites for triaxial test loess sampling on the Banks Peninsula were detailed in this chapter. They are: 1) Moncks Spur (in situ primary air fall loess); 2) Stonehaven Subdivision (valley fill loess-colluvium); 3) Worsleys Spur (primary in situ air fall loess); and 4) Duvauchelle (in situ primary air fall loess).

2. Due to the difficulties of sampling Banks Peninsula loess because of its dry and brittle nature, a field sampling technique had to be developed in order to produce intact specimens for triaxial testing.This method involved hand digging small sample pits in the larger machine dug test pit, (excavated specifically for sampling), filling them with water, and waiting for that water to infiltrate into the underlying soil. This made the loess soft enough to take samples that would stay intact until triaxial testing took place. Samples were extracted from the soil using 35mm stainless steel tube samplers that could take samples that were at least 200mm long.

3. Geotechnical comparisons for soil properties describe Banks Peninsula Loess as a low plasticity silt. Duvauchelle Loess has more clay and plasticity than the others do whereas Worsley spur has the lowest plasticity and the highest sand content. Dry densities range from 1.56 to 1.70 t/m^3.

CHAPTER 4

Triaxial Test Results for Banks Peninsula Untreated Loess

4.1 Introduction

Triaxial test results and analysis for Moncks Spur, Stonehaven Subdivision, Worsleys Spur, Whaka Terrace (untreated samples) and Duvauchelle are presented in this chapter. Results and analysis methods are discussed, and compared to other research that was outlined in Chapter 2. Laboratory preparation methods for water content manipulation in the triaxial test sample are also outlined in this chapter. The primary aims of this chapter are to:

1. Present c and ϕ data for each nominated water content for all field sites
2. Synthesise this data into graph and table form
3. Analyse trends for shear strength parameters versus water content, geography, soil properties, and compare these trends to literature outlined in chapter 2
4. Interpret trends to establish shear strength parameters for all field sites

4.2 Laboratory sample preparation

Considering the objective of this thesis was to measure soil shear strength at differing water contents, the sample preparation method was the most difficult and critical task to master. Nominated water contents were estimated using NZS 4402. The first step, as has been mentioned before, was to make sure that samples collected in the field were wet enough so as not to suffer from brittle fracture, either in the stainless steel tubes whilst being pushed into the ground or upon extrusion. Theory used to achieve water content manipulation was in part taken from Kane's (1968) work with North American Loess, which has been discussed previously in Chapter 2. Kane's drying and rewrapping method was used but not his wet-up process, which was to spray the sample with an amount of water that would achieve the nominated water content. The modified technique for wetting is presented below and was used for all samples from all sites. All soil samples were saturated to “as wet as possible” and then air dried to achieve a nominated water content. Saturation was attained through:

1. covering the ends of the soil (inside the stainless steel tube) with filter paper;
2. packing the tube with steel wool so that the filter paper could be held tight against the sample;
3. covering the ends of the tube with a nylon filter fabric held in place by rubber band;
4. measuring the weight of sample plus tube at constant intervals of one day and once no increase in weight was observed the samples were interpreted to be “as wet as possible”. This levelling out of water uptake in the sample normally took about one week, after which no change was seen to occur even in samples that had been immersed for more than two months.

To achieve a nominated water content percentage (NWCP) in the loess sample after saturation and extrusion from the stainless steel tube, 5 steps had to be followed:

1. The extruded sample was measured so that a sample length of about 100mm could be taken. The sample was then trimmed symmetrically to achieve this;
2. The sample (S) was then weighed and wrapped in glad wrap and again in tin foil and placed in an airtight container so that there would be no water loss;
3. The end pieces (EP) resulting from the trimming were dried for water content measurement (where water content equals the weight of water divided by the wet weight of the sample, this is not NZS 4402, but is a better way to estimate water mass in the trimmed sample);
4. The water contents of the two end pieces (EP) were then averaged and the resulting figure became the water content percentage (WCP) estimate for the trimmed sample (S);
5. This water content percentage (WCP) of the sample (S) was converted to grams of water and was then subtracted from the samples weight to get the dry weight of the sample (dS);

6. The dry weight (dS) was then multiplied by the nominated water content percentage (NWCP) and then this figure was added to the dry sample weight, which equals the sample weight at the nominated water content (SWNWCP);
7. The sample was then unwrapped and air dried till the figure (SWNWCP) above was achieved;
8. The sample was then rewrapped and left for at least a week before triaxial testing took place, so that the water left in the sample could equilibrate.
9. After triaxial testing was conducted on the sample it was weighed for water content by the NZS 4402 standard method of dividing water mass by dry mass of solid.

Although this is a fairly rudimentary preparation method to achieve a nominated water content, water contents measured were normally within approximately +/- 1.5% of the nominated water content.

4.3 Basics of Data Computation

4.3.1 Degree of Saturation data

The degree of saturation was calculated for all test samples, so that loess from different sites could be compared. Comparison of soils using water content only may be incorrect; this is because different soils generally have different porosities and void ratios giving rise to different water contents at saturation. Following from this, soil behaviour at the same water content may be completely different, especially with respect to shear strength.

The degree of saturation in samples prior to triaxial testing was determined by the calculation of four soil properties first, (assuming that there was no shrinkage from 100% saturation) they are as follows:

1. Dry density (ρ_d), by dividing the dry mass of soil (M_s) after testing by the volume of soil before testing (V).

$$\boxed{(\rho_d) = M_s/V} \qquad (4.4)$$

2. Volume of solids (V_s), this was achieved by dividing the dry mass (M_s) of soil by the particle density (specific gravity or G) of loess, which is assumed to be 2650kg/m^3 as this is typical for a quartz rich aeolian deposit.

$$\boxed{V_s = M_s/G} \tag{4.5}$$

3. Volume of voids (V_v) by subtracting the volume of solids (V_s) from the volume of soil (V).

$$\boxed{V_v = V-V_s} \tag{4.6}$$

4. Volume of void water (V_w) by measuring the mass water (assuming the specific gravity of water is 1000kg/m^3).

The degree of saturation (S) is then calculated by dividing the void volume (mass) of water by the volume of voids.

$$\boxed{S = V_w/V_v} \tag{4.7}$$

Saturations were then graphed against water contents so that a relationship could be calculated, which were all approximately linear (see appendix 7 for graphs and relations). Calculation using the linear relationship estimates the water content at full saturation, which means that the x-axis for saturation can be displayed at exactly the same length as for water content, so that water content trends and saturation trends can be compared in the same graphical space (cohesion-angle of internal friction-water content-degree of saturation space). All of the data that is part of the above method is presented in Appendix 1 for all sites.

4.3.2 Cohesion and angle of internal friction data

Mohr-Coulomb failure envelopes are conventionally calculated by applying a line of best fit tangential to Mohr circles on a τ-σ plot (T-S plot), which was presented in figure 2.4. However, as Mohr circles for separate triaxial tests display a large amount of scatter (even if soil properties are identical) there is an accompanying large amount of subjectivity when applying a line to those Mohr circles. Figure 4.1 shows the Mohr circles of stress for each test conducted at an average water content of 16.4% for Moncks Spur Loess, and it easy to see the complications of trying to fit a tangential line of best fit to all circles. To reduce this subjectivity a simple geometric relationship recommended by Dr Kevin McManus (University of Canterbury) can be used using the points of maximum shear stress, $\sigma_1 - \sigma_3/2$ (top of the Mohr circle of stress) to get the Mohr-Coulomb failure envelope. An example from this thesis' results (Figure 4.1) is used to explain the method. The steps required to calculate c and ϕ for Moncks Spur Loess at 16.4% water content are as follows:

1. Points of maximum shear stress and corresponding normal strass are graphed on a t-s plot (test 10B-10J)
2. A linear relationship is calculated from these graphed points. For Moncks Spur Loess at 16.4% water content this relationship has an r^2 of 0.90, showing reasonable scatter of points
3. A linear equation (y=mx+c) is formed from this relationship and in this case it is:

   $$Y=0.64X-8.71$$

4. Angle of internal friction is then calculated by equation 4.8:

   $$\boxed{\sin\phi = \tan\alpha} \tag{4.8}$$

   importing the gradient from point 3 and rearranging equation 4.1 we get:

   $$\sin^{-1}0.64 = 39.6°$$

   39.6° is the angle of internal friction
5. Cohesion is calculated by equation 4.9:

   $$\boxed{c = a/\cos\phi} \tag{4.9}$$

   however, in the case of Moncks Spur Loess at 16.4% water content the intercept a is negative. Negative cohesion is impossible so this has been interpreted to equal zero cohesion
6. Finally a linear equation (y=mx+c) (4.10) for the Mohr-Coulomb envelope is calculated from the tangent of ϕ so that the failure envelope can be presented graphically (figure 4.2):

   $$\boxed{Y = \tan\phi + a/\cos\phi} \tag{4.10}$$

   For Moncks Spur Loess at 16.4% water content this is:

   $$Y=0.82 + 0$$

This method has been used for all nominated water contents in this thesis.

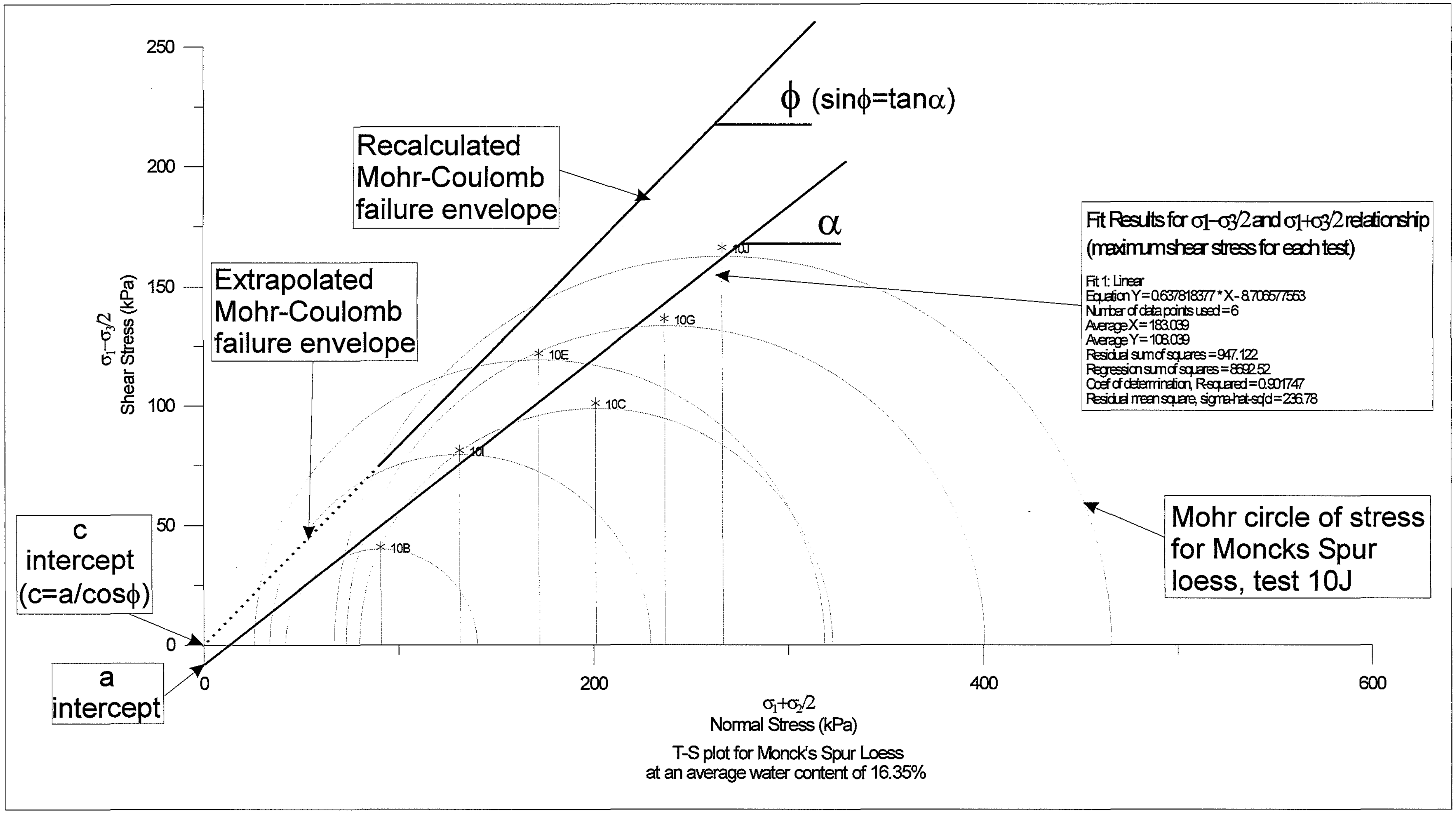


Figure 4.1 Geometric relations between normal/shear stress and the Mohr-Coulomb failure envelope

4.4 Triaxial test results for Banks Peninsula Loess

A summary of triaxial test results for cohesion and angle of internal friction are presented in figure 4.2 as graphs of Mohr-Coulomb failure envelopes, and a summary table (Table 4.1) presents accompanying results.

4.4.1 Moncks Spur Loess (Figure 4.2a)

A total of 51 unconsolidated undrained triaxial tests were conducted on Moncks Spur P-layer Loess, of which 27 were failures. Eleven were tested with a faulty load cell, 9 were tested properly but water content measurements were lost. Testing failures were reduced considerably for other sites as Moncks Spur was the first site to be tested. Although moisture increments were targeted at 2% water content intervals from 6% to 18%, in practice this was not possible and tests were therefore grouped into water content ranges of 6-9%, 9-12%, 12-15% and 15+%. The average water content was taken to give a value for that specific water content group, which were 7.79%, 11.90%, 13.87%, and 16.35% (see Appendix 1 for water contents and test data for all samples from all sites). For cohesion, as water content rises from 8% it increases, reaches a maximum (45.6 kPa) at 12% and drops rapidly to zero cohesion at approximately 16% water content. Conversely angle of internal friction follows the opposite trend with a minimum of 30° occurring at approximately 13% water content. (For all calculations of cohesion and angle of internal frictions please refer to Appendix 3: T-S plots for all sites)

4.4.2 Stonehaven Subdivision Loess (Figure 4.2b)

Forty eight samples were collected from the Stonehaven Subdivision test pit. The testing regime for soil sampling of Moncks Spur loess did not prove successful at 2% water content increments, so it was decided to split the 48 samples into four groups of twelve for water content preparation, which were 6%, 10%, 14% and "as wet as possible". During preparation three samples from each of the four groups were broken and unable to be tested. The actual averaged water contents were 6.9%, 9.4%, 17.3% and 22.9%. At approximately 7.5% water content cohesion increases from 0 kPa and angle of internal friction drops to 51.8°. Maximum cohesion value reaches 11.7 kPa at 17.3%, and then drops off as water content increases. The angle of internal friction levels off at 13° at approximately 23% water content, and saturation is not reached until 29%.

4.4.3 Partial results for Worsleys Spur (Figure 4.2c)

This was the last field site to be sampled and tested. 48 triaxial test tube samples were collected at Worsleys Spur. Four nominated water contents (6%, 12%, 15% and "as wet as possible") were targeted for variance, and no testing failures occurred. Nominated water contents 10 and 14% were changed to 12 and 15% to target shear strength parameters at the brittle-ductile boundary, as it was thought that this is where it occurred. However, data contained in the Geomechanics Laboratory computer 1 was lost due to a computer hard drive failure. As no back-up was made, only the 6% targets and 12% targets were saved. The majority of stress-strain graphs for all tests sites were also lost. Samples were accidentally dried past the nominated water and the average water contents for both nominations were out by 4% the result being 2.38% and 8.4% respectively. Cohesion rises with water content starting at 2.5% and reaching a maximum of 50 kPa at 8%, conversely angle of internal friction drops from 63° to 32° respectively.

4.4.4 Duvauchelle Loess (Figure 4.2d)

Of the 48 triaxial tube samples collected 31 were tested. It was hoped that 4 sets of 12 samples could be tested at different water contents (6%, 10%, 14% and as saturated as possible). However, the set of 12 intended for saturation were not investigated because it proved impossible to increase the water content of the material above field water contents. The average water contents for each nominated group were 8.41%, 11.45% and 16.11% respectively. Cohesion is 219 kPa at a water content of 8.4%, but drops off steeply to zero cohesion at 11.5%, and stays at zero until 16.1%. The angle of internal friction increases to a maximum of 53° at 11.5% water and then decreases to 42° at 16.1%.

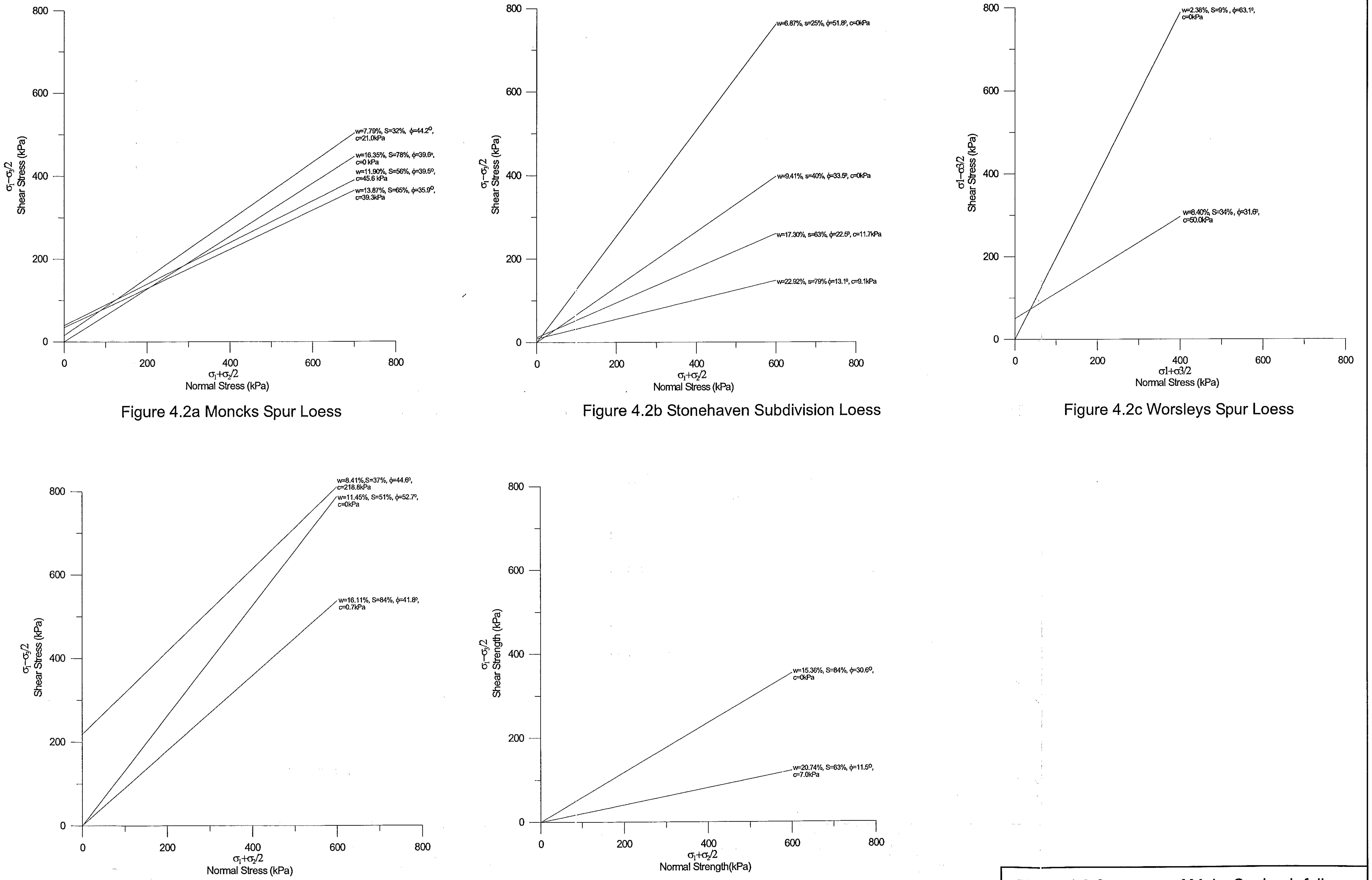


Figure 4.2a Moncks Spur Loess

Figure 4.2b Stonehaven Subdivision Loess

Figure 4.2c Worsleys Spur Loess

Figure 4.2d Duvauchelle Loess

Figure 4.2e Whaka Terrace Loess

Figure 4.2 Summary of Mohr-Coulomb failure envelopes for Banks Peninsula Loess samples

4.4.5 Whaka Terrace untreated loess (Figure 4.2e)

12 samples were collected from Whaka Terrace untreated loess fill to act as a control for the samples treated with hydrated lime. The sample regime did not conform to the other sites as this project had different aims. However, the data is useful for this thesis' primary aim. The 12 samples were subdivided into two nominated water contents as were the treated samples. Targets for water contents were 15% and "as wet as possible", the actual water contents were measured at 15.36% and 20.74% respectively. Analysis of figure 4.2e shows that as water content increases from 15.4% cohesion increases from 0 to 11.5 kPa at 21% water content; in the same water content range angle of internal friction decreases from 31° to 7°.

Results for all sites are summarised in table 4.1 below

Field site	Nominated water contents	Tested water content	Saturation	Cohesion	Angle of internal friction
Moncks Spur	8%	7.79%	32%	21	44
Loess	12%	11.90%	56%	46	40
	14%	13.87%	65%	39	36
	16%	16.35%	78%	0	40
Stonehaven	6%	6.87%	25%	0	52
Subdivision	10%	9.41%	40%	0	34
Loess-Colluvium	14%	17.30%	63%	11	23
	"as wet as posible"	22.92%	79%	9	13
Worsleys Spur	6%	2.38%	9%	0	63
Loess	12%	8.40	34%	50	32
Duvauchelle	6%	8.41%	37%	218	45
Loess	10%	11.45%	51%	0	53
	"as wet as possible'	16.11%	84%	1	42
Whaka Terrace	15%	15.36%	68%	0	31
Loess Fill	"as wet as possible"	20.74%	84%	11	7

Table 4.1 Triaxial test results for all filed sites and nominated water contents

4.5 Data and trend analysis

4.5.1 Water content and degree of saturation trends

4.5.1.1 Moncks Spur Loess

Analysis of figure 4.3a for Moncks Spur Loess reveals two important trends:

1) With an increase of water content an increase of cohesion to a maxima and then a decrease;
2) The inverse of the preceding trend for the angle of internal friction, which is, with an increase in water content there is a decrease of angle of internal to a minima and then an increase.

This is not the trend that was thought to occur for Port Hills Loess, as it was expected (and has been recorded in past literature) that cohesions are somewhat higher than this for loess tending towards dryness. Maximum cohesion and minimum angle of internal friction occur at approximately the same water content. Comparison of saturation and water content trends for Moncks Spur loess shows a shift to the right of water content trends for both cohesion and angle of internal friction. This suggests that there is a non-linear relationship between water content and degree of saturation, or the water content calculated for 100% saturation is incorrect. If the relationship is non-linear then this could mean that the total soil volume changes with varying water content, suggesting that shrinkage and expansion occurs in the soil with varying water content.

4.5.1.2 Stonehaven Subdivision Loess

Part of the same trend seen in Moncks Spur Loess for cohesion and angle of internal friction can be seen in Stonehaven Subdivision Loess (Figure 4.3b). This is evident when cohesion and angle of internal friction increases and decreases (for increasing water content) respectively until a maxima or minima is reached. There are no trends after this point as no data has been recorded for this water content range. The preceding trends also occur when cohesion and angle of internal friction are compared to degree of saturation. Trends for saturation and water content almost overlay each other suggesting that a there are no volume changes in the soil with increasing water content.

4.5.1.3 Worsleys Spur Loess

A partial trend is shown in figure 4.3c for Worsleys Spur Loess because of the hard drive failure discussed in section 4.4.3. However, results also suggest a trend similar to the two sites discussed above. It can be inferred that cohesion would be very much reduced at close to or at saturation, and that the trend would result in something similar to the Moncks Spur site. Similar behaviour occurs for saturation and water content comparisons to Stonehaven Subdivision Loess as they virtually overlay each other in the same graph space, suggesting that small volume changes occur within the soil with increasing water content.

4.5.1.4 Duvauchelle Loess

Qualitative analysis (as in all analyses carried out in section 4.5) of Duvauchelle loess (figure 4.3d) shows a slight inverse exponential relationship between water and cohesion, as water content increases cohesion decreases. Analysis of the angle of internal friction produces a trend which increases to maxima of 53° and then a decrease. Saturations follow the same trends as water contents but are shifted to the left (opposite to Moncks Spur) suggesting that a total soil volume change occurs as water content increases, or that water content (17.61%) at saturation has been calculated wrong and is lower than what it should be.

4.5.1.5 Whaka Terrace Loess (figure 4.3e)

Trends and relationships for Whaka Terrace Loess are partially analysed as only two nominated water contents were tested. As water content increases from approximately 15%, cohesion increases and angle of internal friction decreases. Saturation trends practically overlay water content, but there is a slight shift to the left, this could be the result of the water content at saturation being calculated too high.

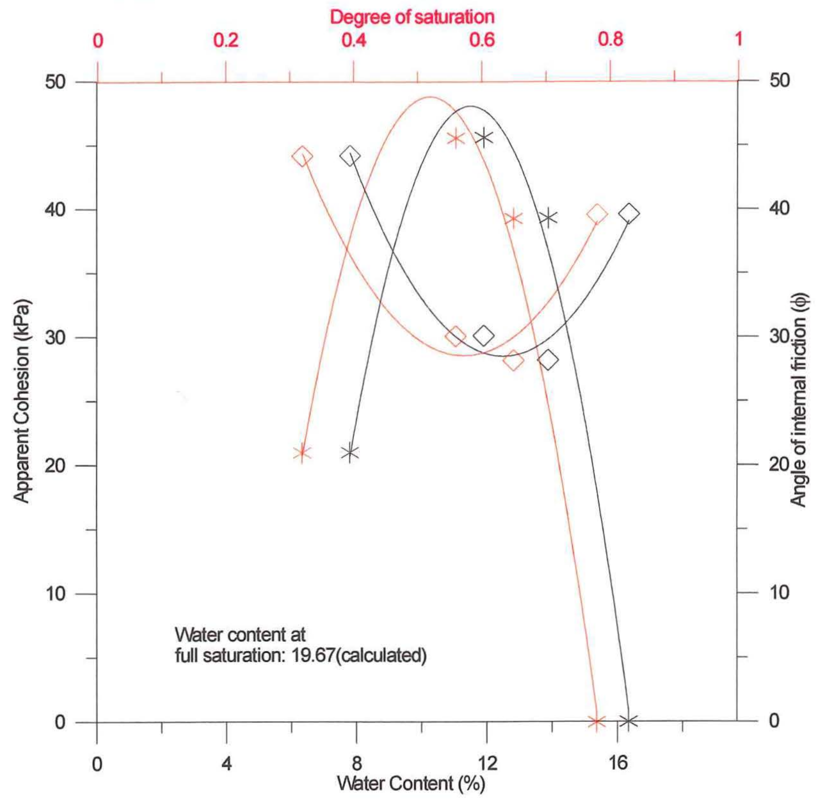


Figure 4.3a Moncks Spur Loess

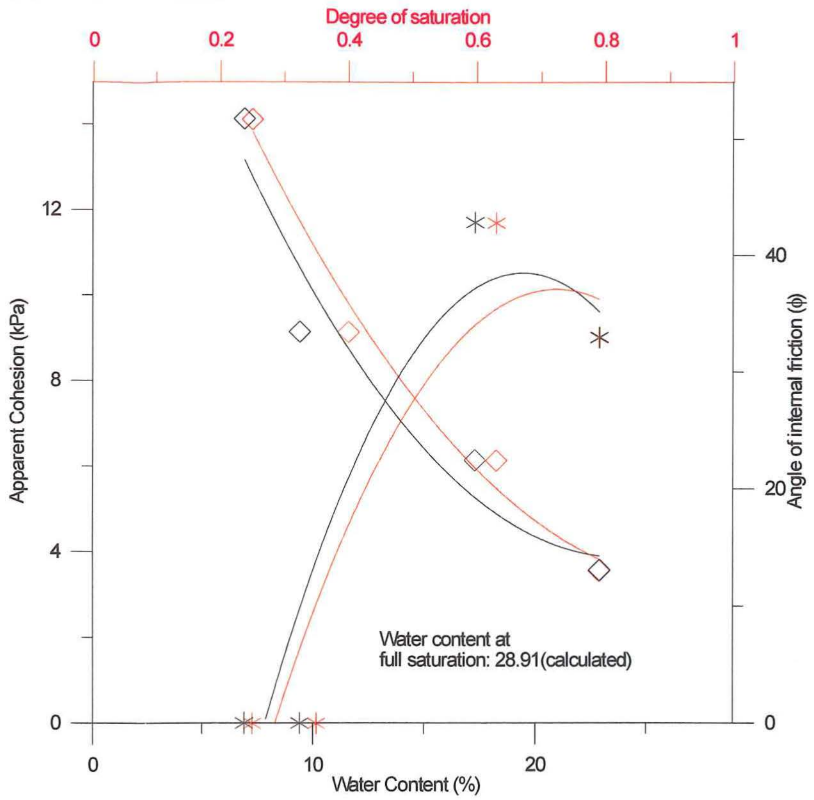


Figure 4.3b Stonehaven Subdivision loess-colluvium

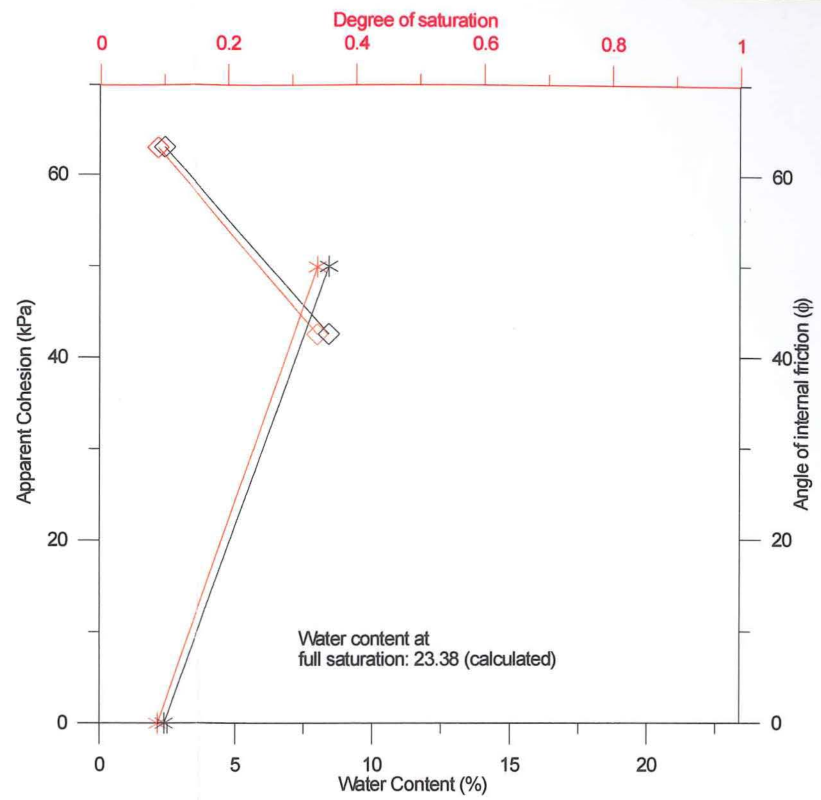


Figure 4.3c Worsleys Spur Loess

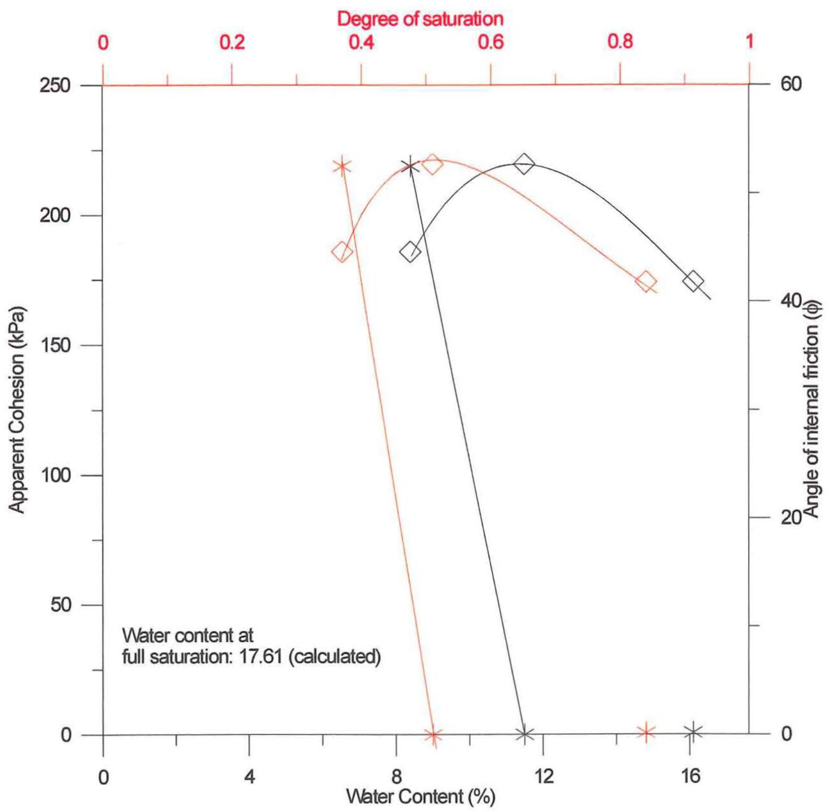


Figure 4.3d Duvauchelle Loess

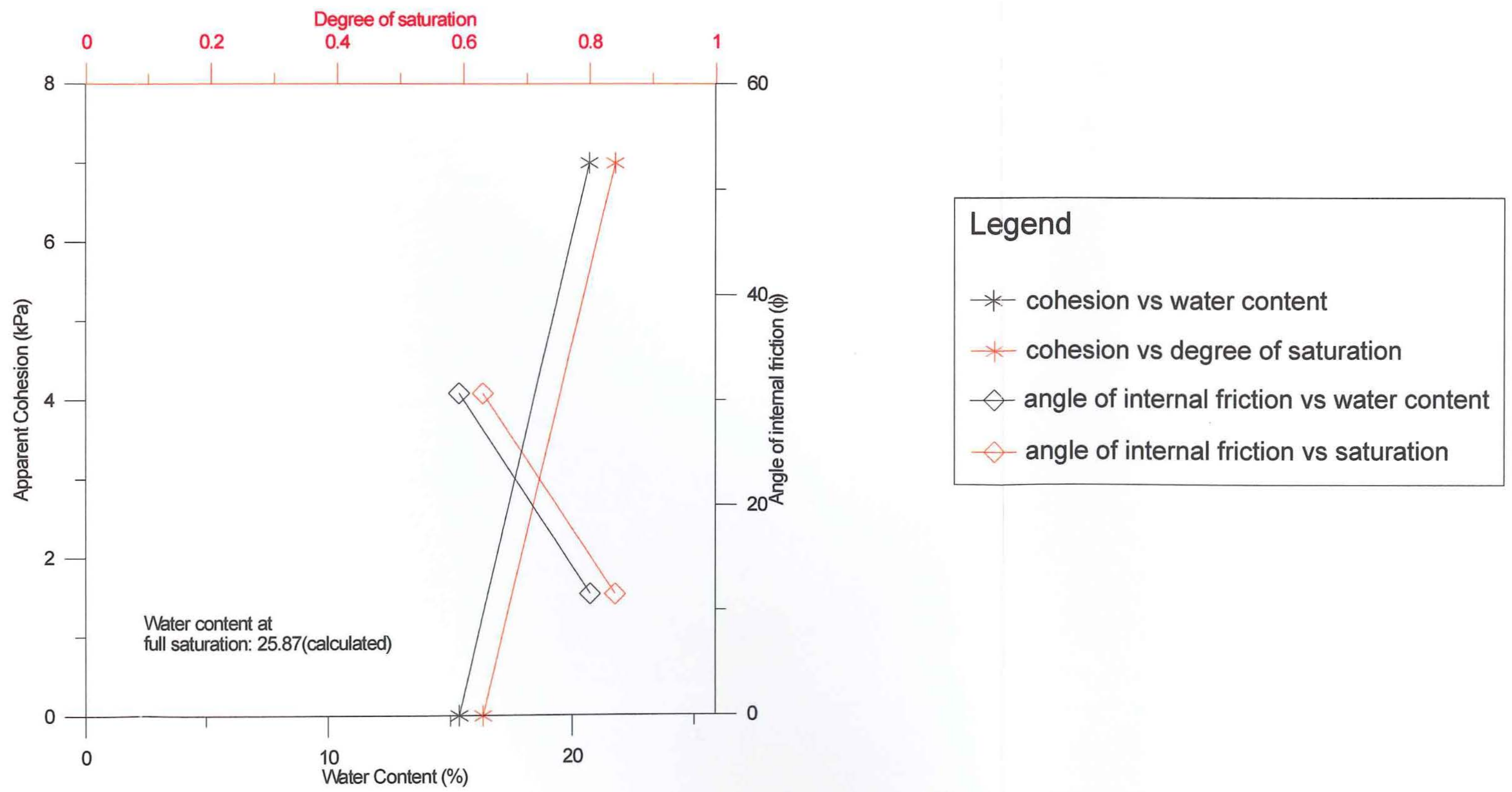


Figure 4.3e Whaka Terrace untreated loess fill

Figure 4.3 Water content and saturation relations for all field sites

4.5.2 Other trends and comparisons

4.5.2.1 Port Hills site comparisons

Four of the five sites (Moncks Spur, Stonehaven Subdivision, Whaka Terrace and Worsleys Spur) tested are evenly spaced along the northern side of the Port Hills (refer to figure 1.1). However, two of the sites represent redeposited loess in the form of a fill and a loess-colluvium, whilst the other two are in situ primary airfall loess deposits. Duvauchelle (primary airfall in situ loess), being relatively remote from all other sites, has a different rainfall, soil layering (Griffiths, 1973), clay content, and is therefore distinct from all other sites.

Analysis of Figure 4.4a and 4.4c shows one similarity for the sites across the Port Hills northern range front and that is an increase of cohesion and decrease in angle of internal friction for lower water contents. Differences occur at the water content these trends are seen. For the fills/colluviums this behaviour is observed to start at around 10-15% and for the primary airfall loess' this occurs at a water content of around 6-8%.

For angle of internal friction all of the northern Port Hills sites show similar trends of increasing water content with decreasing angle of internal friction. Figure 4.4b shows the trend overlapping between a degree of saturation of 20 to 60%. For 4.4d the trend is slightly less pronounced and occurs at water contents between 5 and 12%. Trends for Whaka Terrace Loess occur at even higher water contents and saturations than all others, starting at 15% water content and 65% saturation.

Another correlation, which can be seen specifically in Port Hills Loess, is that shear strength parameters for Worsleys Spur Loess appear to be far higher than Monck's Spur Loess. Both are of the same facies (Birdlings Flat Loess) as described by Griffiths (1973), but Worsleys Spur has a far higher clay and sand content at the expense of silt. Higher clay content could be responsible for the seemingly higher cohesion (seemingly higher because the data set is not complete for Worsleys Spur); however, Worsleys Spur has the lowest measured plasticity index. Conclusions are difficult to draw as no clay mineralogy has been determined for any of the sites.

Both the fill and colluvium of Whaka Terrace and Stonehaven Subdivision respectively show the least amount of cohesion. This could be due to the remoulding effects of redeposition.

4.5.2.2 Port Hills and Duvauchelle Loess comparisons

For ease of comparison Table 4.2 (taken from table 3.2) has been placed onto figure 4.4 so that soil properties can be referred to easily. Whaka terrace results have been added as well as properties from Loess reviewed in Chapter 2.

In comparing shear strength parameters in terms of soil properties one comparison stands out and that is the difference between Port Hills Loess and Duvauchelle Loess. Duvauchelle Loess displays a far higher cohesion value (219 kPa) it also has a somewhat higher density and smaller porosity, and, by far the highest clay content. However the soil also reaches 0 cohesion with the least amount of water content (~11%) and saturation (0.51). Another peculiar property associated with the low porosity is its low water content when saturated, although the liquid limit is the highest of all soils.

4.5.2.3 Comparisons with previous literature

Comparisons for Banks Peninsula Loess centre on McDowell's (1988) work on Coleridge Terrace Loess in Lyttelton. Under Griffiths (1973) type classification and loess distribution map (Figure 1.4) it would appear that McDowell's Coleridge Terrace Loess is Birdlings Flat Loess, although McDowell does not make this distinction. The tested loess is a buried fragipan ("C" layer loess) and is not directly comparable to the "P-Layer" loess of Port Hills sites, although, it is important to note that McDowell uses exactly the same calculation method as that presented in

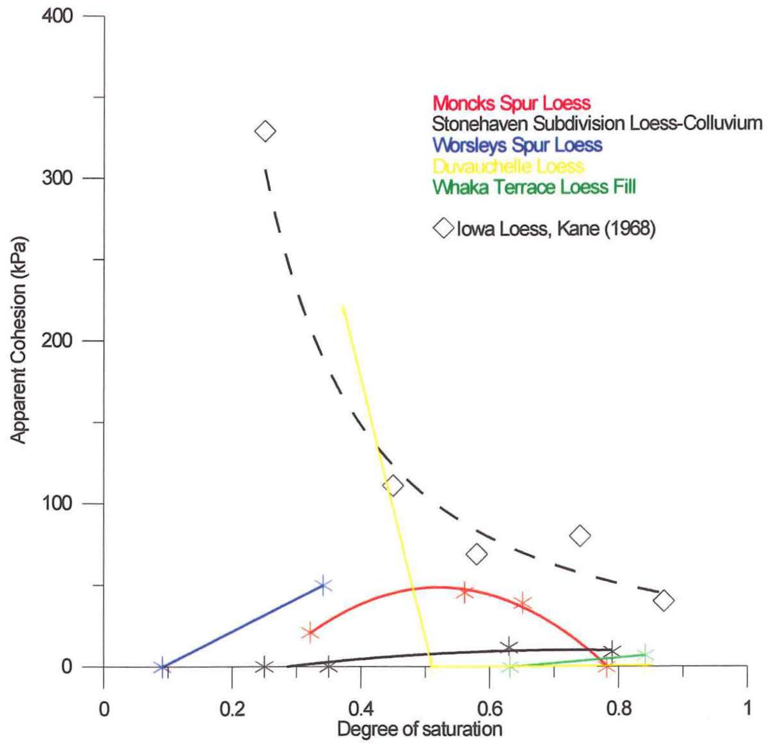


Figure 4.4a Cohesion and degree of saturation relations

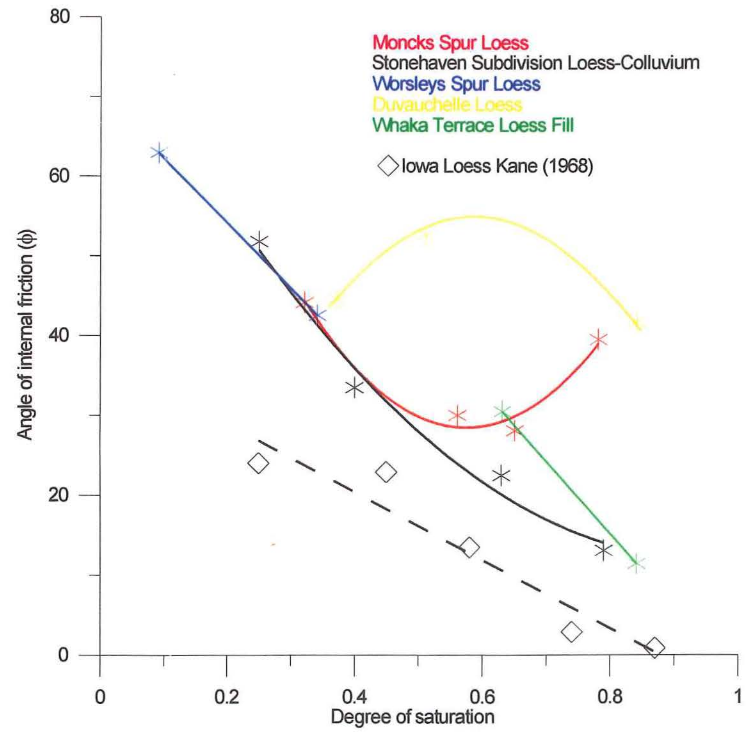


Figure 4.4b Angle of internal friction and degree of saturation relations

Field sites	Sand	Silt	Clay	PL	LL	PI	γd	n	e
Moncks Spur Loess	14.19	76.08	9.73	26.10	20.02	6.08	1.68	0.42	0.59
Stonehaven Subdivision Loess	17.36	77.58	5.20	26.38	10.31	16.07	1.53	0.42	0.74
Worsleys Spur Loess	25.64	62.05	12.32	22.94	18.79	4.11	1.56	0.42	0.71
Duvauchelle Loess	15.11	65.41	19.47	25.25	16.5	8.79	1.70	0.36	0.56
Whaka Terrace untreated	15.51	72.15	12.34	22.73	16.27	6.52	1.61	0.40	0.65
Literature									
Loess Iowa Loes Kane (1968)			19	35	24	11	1.41		0.95
Colleridge Terrace fragipan McDowell(1988)	12	69	19	24	16	8	1.88		

Table 4.2 Summary of soil properties for field sites and reviewed literature

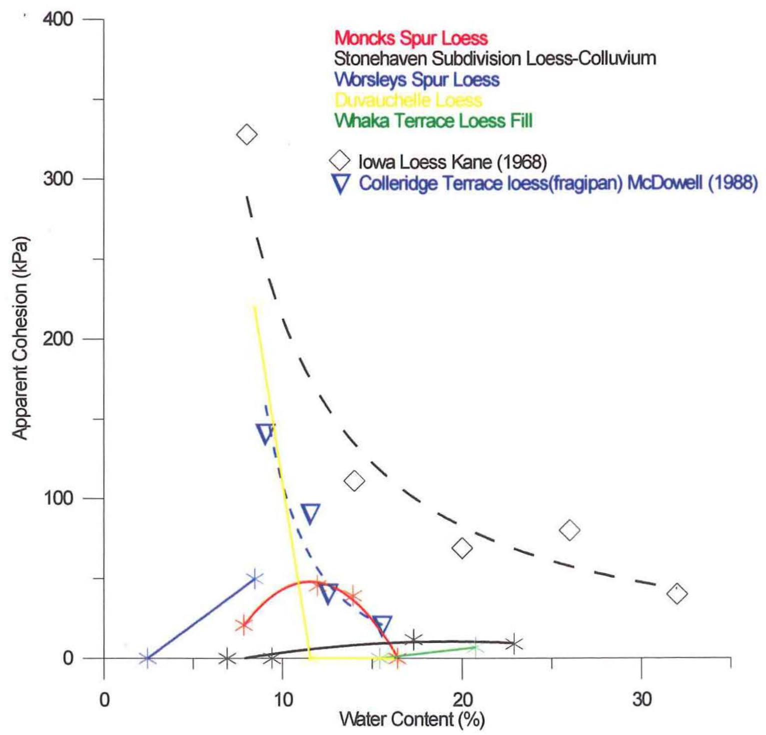


Figure 4.4c Cohesion and water content relations

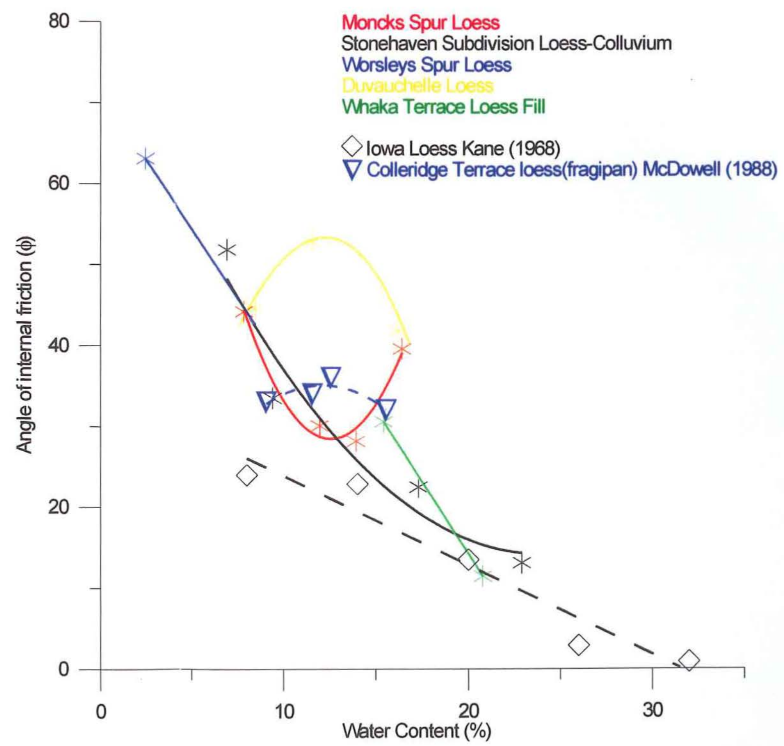


Figure 4.4d Angle of internal friction and degree of saturation relations

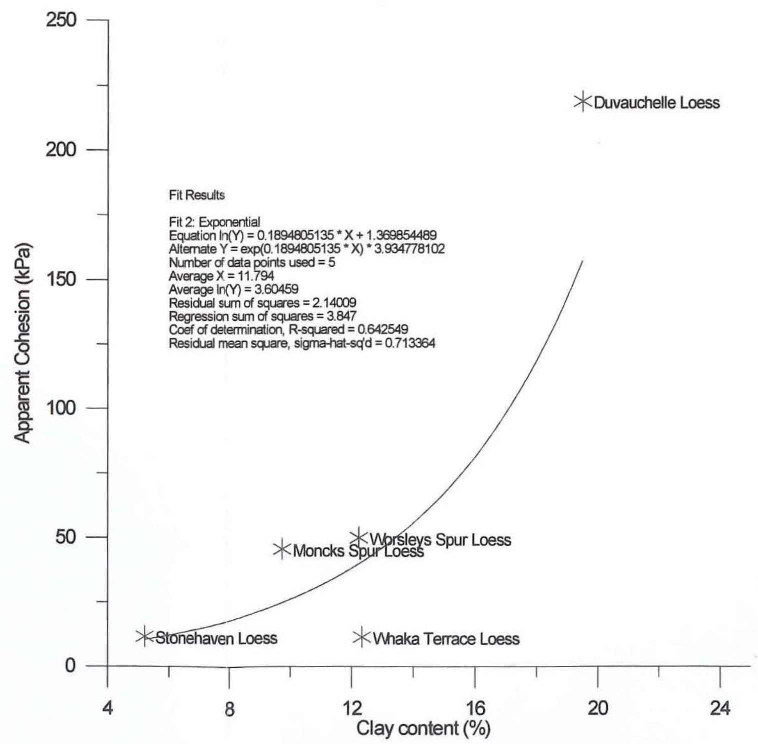


Figure 4.4e Clay content relations

Figure 4.4 Site locations, soil properties and literature comparisons for Banks Peninsula Loess section 4.3.2, Calculation of cohesion and angle of internal friction. Coleridge Terrace angle of internal friction shows little or no effect with increasing water content and stays approximately in the mid to low 30's. Comparison of Coleridge Terrace loess with the other Port Hills sites shows there is marked difference in the shear strength properties of "C" layer loess and "P" layer loess.

The only comparison for shear strength parameters, which can be made for overseas loess is the results provided by Kane (1968), these are shown on figure 4.4. Kane's relationship shows decreasing cohesion and angle of internal friction with increasing water content. Cohesion follows an exponential decrease, whilst angle of internal friction shows a linear decrease. Cohesions are generally higher than Port Hills Loess and angle of internal frictions are lower than Port Hills Loess. This is probably due to Iowa's higher clay content (Figure 4.4e) and plasticity index (Table 4.2). Conversely there are similarities between Iowa Loess and Duvauchelle Loess especially in the case of cohesion, which show similar exponential decreases and both have higher cohesion maxima (219 kPa for Duvauchelle loess at 8.4% water content and 329 kPa for Iowa Loess at 8% water content).

4.6 Discussion

4.6.1 Shear strength dependence

From the analyses given throughout section 4.5 (Data Analysis) it can be concluded that shear strength of Banks Peninsula Loess is dependant on water content. All areas show a variation (both increasing and decreasing) of cohesion and angle of internal friction to increasing water content.

4.6.2 Type and origin of shear strength dependence

Various shear strength comparisons and trends have been presented above they are water content, site locations and comparisons with previous literature. A clear distinction can be made between Port Hills Loess and Akaroa Loess because of differences in recorded cohesions and angle of internal frictions, distance in site locations, clay content and rainfall environment.

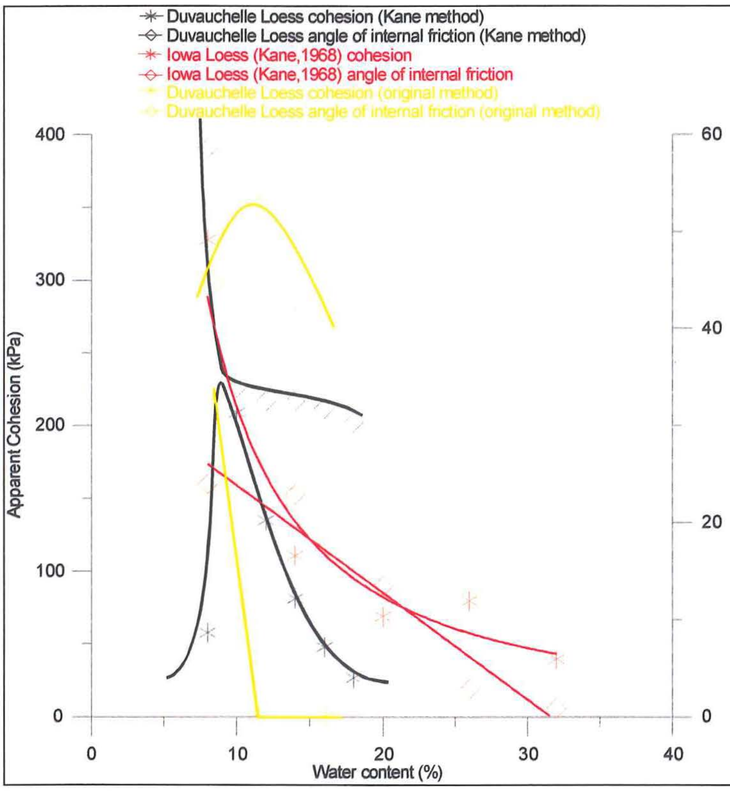
4.6.3 Port Hills Loess

For Port Hills Loess there is similarity in shear strength behaviour as all show increases with cohesion as water content reaches a maximum, and then decrease in cohesion beyond this. This can be interpreted as a zone of cohesion, which happens between saturations of 10% and 80%. Water content at saturation is not consistent, making comparisons in water content itself difficult. This has not been observed in previous research. It could be argued that this new trend is an artefact of the calculation method used for cohesion and angle of internal friction. However, this trend is not prevalent in Duvauchelle Loess and the method has been used previously by McDowell (1988) with entirely different results (although a different soil layer was tested). Angle of internal friction appears to decrease linearly with increasing water content in water content ranges tested for all sites except Moncks Spur. The cohesion trends could possibly be attributed to soil suction (negative pore water pressures) effects as there is no cohesion present when the soils are almost dry. Further tests on loess soils, which are dried to lower water contents than in this thesis need to be carried out to prove this trend.

4.6.4 Duvauchelle Loess

For Duvauchelle Loess, behaviour is quite different as there appears to be an exponential decrease in cohesion with increasing water content similar to what Kane (1968) found. Duvauchelle Loess also records the highest cohesion (219 kPa) and has the highest clay content and plasticity of any of the Banks Peninsula sites. It was decided to use exactly the same method as Kane used to calculate cohesion and angle of internal friction. This involves the formulation of a relationship of shear stress ($\sigma_1 - \sigma_3/2$) against water content for each cell pressure (Kane used eight separate cell pressures, this project used three). Followed by nominating a water content and interpolating this nominated water content on the new shear stress-water content relationship to get a new shear stress value

($\sigma_1-\sigma_3/2$) and then calculating the corresponding normal stress value ($\sigma_1+\sigma_3/2$). The new stress values are graphed on a T-S plot and calculations are made for cohesion and internal friction as outlined in section 4.3.2. This was attempted for all other sites, but the calculation method produced nonsensical results in the form of very low (<10kPa and 108) cohesions and angles of internal frictions for all water contents.



Duvauchelle Loess (Kane method)		
Water%	Cohesion	Angle of internal friction
8	57.6	58.6
10	208.6	33.9
12	134.8	32.5
14	81.1	32.5
16	47.9	31.7
18	26.6	30.3
Iowa Loess (Kane, 1968)		
8	328	24
14	111	22.9
20	69	13.5
26	80	2.9
32	40	0.9
Duvauchelle Loess - original calculation		
8.41	218	45
11.45	0	53
16.11	1	42

Figure 4.5 Recalculated shear strength relations for Duvauchelle Loess

Water contents using this method can be nominated at any percentage, and it was thought that constraining the nominated water contents are best constrained but actual water contents testing was conducted at. However, as any water content could be chosen to calculate cohesion and angle of internal friction it was decided to choose subdivisions of 2% so that trends would be shown in better resolution than what averages of actual water contents could achieve. Duvauchelle Loess shear strength parameters calculated from this method produces an exponential decrease with increasing water content similar to the original relationship (given in figure 4.3d), but more closely resembles the relationship produced by Kane (1968) for Iowa state loess (figure 4.5). However, the trend of low cohesion at low water contents that was evident in Port Hills loess is now also present in Duvauchelle loess.

Similarity of cohesion trends for these two loess materials is most likely due to their similar clay contents, which are 19.5% for Duvauchelle Loess and 19% for Iowa Loess, differences could be the result of different clay mineralogies, which have not been explored in this thesis. Results show that zero cohesion is not reached until after 22% water content and that the cohesion for 16% water content is 48 kPa. Angle of internal friction in figure 4.6 now shows a linear decrease as compared to figure 4.5d, and no dramatic changes are seen. Figures for 4, 6 and 8% are anomalous and are thought to be a weakness of Kane's (1968) analysis as angle of internal friction is extremely high at these water contents and cohesions are anomalously low.

4.7 Synthesis

1. As shear strength dependence on water content was the prime aim of this thesis, a method whereby water could be varied in the triaxial test specimens so that specific water content groups could be nominated for testing had to be developed. Methodology developed by Kane (1968) was used extensively for this thesis' method, although his "wetting up" procedure was not followed.

2. Nominated water contents for test sites were: 4, 6, 8, 10, 12, 14, 16 and 18 for Moncks Spur loess; 6, 10, 14 and saturated for Stonehaven Subdivision Loess; 6, 10 and 14 for Duvauchelle Loess; 6, 10, 15 and saturated for Worsleys Spur Loess; and 10 and 15% for Whaka Terrace Loess. Actual water contents as measured after triaxial testing were: 7.79, 11.90, 13.87 and 16.35%; 6.87, 9.41, 17.30 and 22.92%; 2.38 and 8.40%; 8.41, 11.45, 16.11%; and 15.36, 20.74 respectively.

3. Mohr coulomb envelopes were calculated using a simple geometric relationship using maximum shear stresses for all triaxial test results

instead of the conventional fit to Mohr circle approach, which would have been too difficult due to scatter.

4. Shear strength dependence on water was established based on trends formulated in water content-saturation-cohesion-angle of internal friction space.

5. A new trend of increasing cohesion and decreasing angle of internal friction with increasing water content was documented for all Port Hills Loess, but was not found for Duvauchelle Loess.

6. Port Hills fills and loess colluviums (Whaka Terrace and Stonehaven Subdivision respectively) have similar shear strength trends and are distinct from primary airfall in situ Port Hills loess' (Moncks Spur and Worsleys Spur), which have higher cohesions.

7. Port Hills Loess is distinct from Duvauchelle Loess based on shear strength relations and a far higher cohesion (219kPa). Clay content could be the reason for Duvauchelle as it recorded a far higher cohesion than Port Hills Loess.

8. Using Kane's (1968) calculation of c and ϕ method Duvauchelle Loess was found to have an exponential decrease in c and a linear decrease in ϕ for increasing water content. The trend of low cohesion with low water contents is shown in this new calculation method.

CHAPTER 5

Lime Stabilisation Trial at Whaka Terrace

5.1 Introduction

Talks with Marton Sinclair (Eliot Sinclair Ltd) highlighted the need for more research into the stabilisation of uncompacted loess fill, which is a common feature of older roads and subdivisions on the Port Hills. It was then decided to research the effects of hydrated lime on uncompacted loess fill at Whaka Terrace, Port Hills on the shear strength parameters c and ϕ. A review of previous research on lime stabilisation of Port Hills Loess found that no triaxial shear testing had been conducted on hydrated lime treated loess. In keeping with this thesis' overall aims, unconsolidated undrained triaxial shear testing was conducted on Whaka Terrace Loess (treated and untreated) at varying water contents. This Chapter represents the secondary aim of this thesis, and will be fulfilled by:

1. Reviewing past literature to formulate a method of lime application to Whaka Terrace uncompacted loess fill
2. Outlining sample preparation, both in the field and laboratory
3. Presenting and analysing results from triaxial shear testing

5.2 Previous Research on Lime Stabilisation.

5.2.1 Evans and Bell (1981)

Evans and Bell (1981) present a paper on the chemical stabilisation of Port Hills Loess in which they discuss two methods of stabilisation. The first is to stabilise loess soils using phosphoric acid (H_3PO_4) and the second is to use hydrated lime ($Ca(OH)_2$). Soils used were collected from Huntsbury Spur (Whaka Terrace is also positioned on this spur) specifically sampled from the "P-layer" (Hughes, 1970).

Both stabilisation chemicals were found to reduce erodability, and dispersiveness, to improve drainage (increased permeability) and unconfined compressive strength, with phosphoric acid providing slightly more strength gain than quick lime. Material was also collected by Evans in 1978 from Glenelg spur. His paper summarises the effect that lime has on untreated loess soils in the following four points, summarised in Evans and Bell (1981) relating to figure 5.1:

1. There is a significant decrease in unconfined compressive strength with increasing initial water content, and the untreated control and 1% lime samples show similar trends.
2. The proportionate increase in unconfined compressive strength with 1% lime addition is greater at water contents wet of optimum moisture content for the untreated soil (14.6%), and decreases with decreasing initial water content.
3. Lime additions at 3% and 5% by weight of dried soil increase the value of σ_{co} above that for 1%, but there is obvious experimental scatter.
4. At the optimum moisture content for the untreated loess, strength gains of two to three times can be expected with lime additions up to 5% (Evans and Bell 1981).

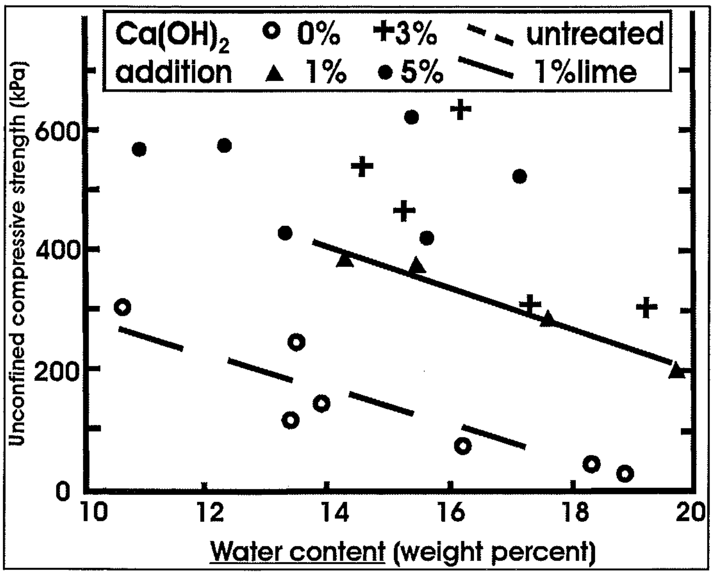


Figure 5.1 Unconfined compressive strength-water content relationships for additions (as weight percent dried soil) of lime to Glenelg Spur Loess (From Evans and Bell, 1981)

With the Hunstbury site two samples produced a maximum unconifined compressive strength of between 800 and 900 kPa with the 5% addition of lime, whilst compacting the samples just wet of optimum moisture practically doubled the strength gain at 5% from approximately 550 kPa to 900 kPa(figure 5.2). Seven day moist curing in a standard proctor mould was the method used for general preparation of the loess soil. In conclusion, Evans and Bell (1981) state that, whilst phosphoric acid displays better stabilising qualities, the use of lime is preferred for safety reasons, and that more research needs to be carried out on the immersed (saturated) strength characteristics of Port Hills Loess.

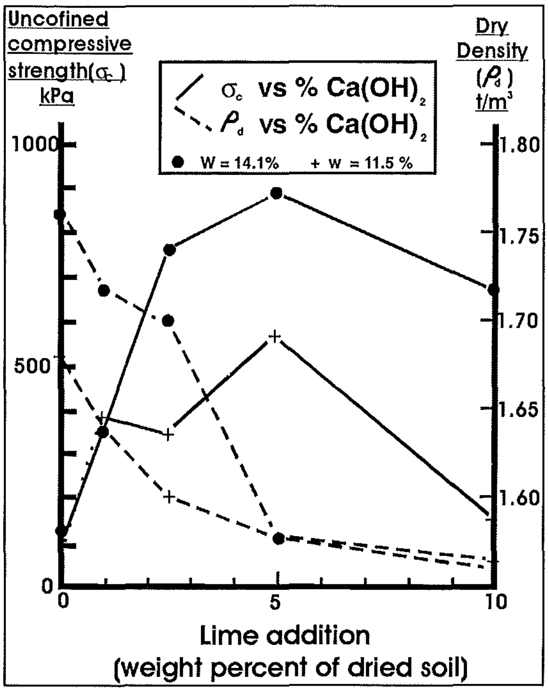


Figure 5.2 Unconfined compressive strength and compacted dry density plots for lime additions to parent loess from Huntsbury Site 2 (From Evans and Bell, 1981)

5.2.2 Glassey (1986)

The second piece of research on loess stabilisation using lime is presented in a thesis by Glassey (1986). Glassey cites six different factors affecting the strength of treated samples, as follows: 1) lime content; 2) type of lime; 3) type of soil; 4) dry density of soil; 5) type of curing; 6) time of curing. Glassey also suggests, in a review of previous literature, that there is a threshold of lime addition above which strength gains decrease, the reason being that the lime has reacted with all of the potential strength-gaining soil particles, and that the leftover lime forms a lubricating gel which in effect reduces the amount of internal shearing resistance that the lime has created.

The unconfined compressive strength test was used by Glassey to evaluate relative strength gains. Glassey tests strength is tested at 0%, 1%, 2.5%, 5%, 7.5% and 10% lime content, and three different curing techniques, which seem to be the main focus of his project. The three techniques for curing were: 1) 14 days moist (wrapped in a plastic bag and placed in the fog room) at 20 ^{O}C and 99% humidity; 2) Seven days moist curing followed by seven days air drying; 3) Seven days moist curing, then repeated 24 hour cycles of wetting and drying. Curing method one was used to simulate field conditions of a buried soil in the field. Curing method two was used to represent conditions faced for soils treated on the Port Hills, and curing method three was used to represent conditions encounted in soils which line drains and small artificial lakes.

Figure 5.3 outlines Glassey's result's for unconfined compressive strength testing of Westmorland subdivision loess. Figure 5.3A shows the relationship when the soil has been moist cured for 14 days; there is a marked increase in compressive strength, however, the increase lessens between 1 and 7.5 per cent. Figure 5.3B also shows a minimum out of compressive strength between 1 and 7.5%, and likewise with the wetting and drying cycles undergone by loess in figure 5.3C. Along with Glassey (1986), Evans and Bell (1981) also report a drop off in compressive strength at about 5% lime addition.

Figure 5.3D provides the most interesting data of all three sets in that compressive strengths from all three curing methods have been placed on one graph, and plotted against water content. From here it is possible to interpret the increased compressive strengths due to lime treatment as a function of water content solely, and to conclude that curing processes may not have any effect on resultant strength at all. Finally, what can be taken from this research is that the addition of 1-2% lime appears to be the optimal addition for strength gain.

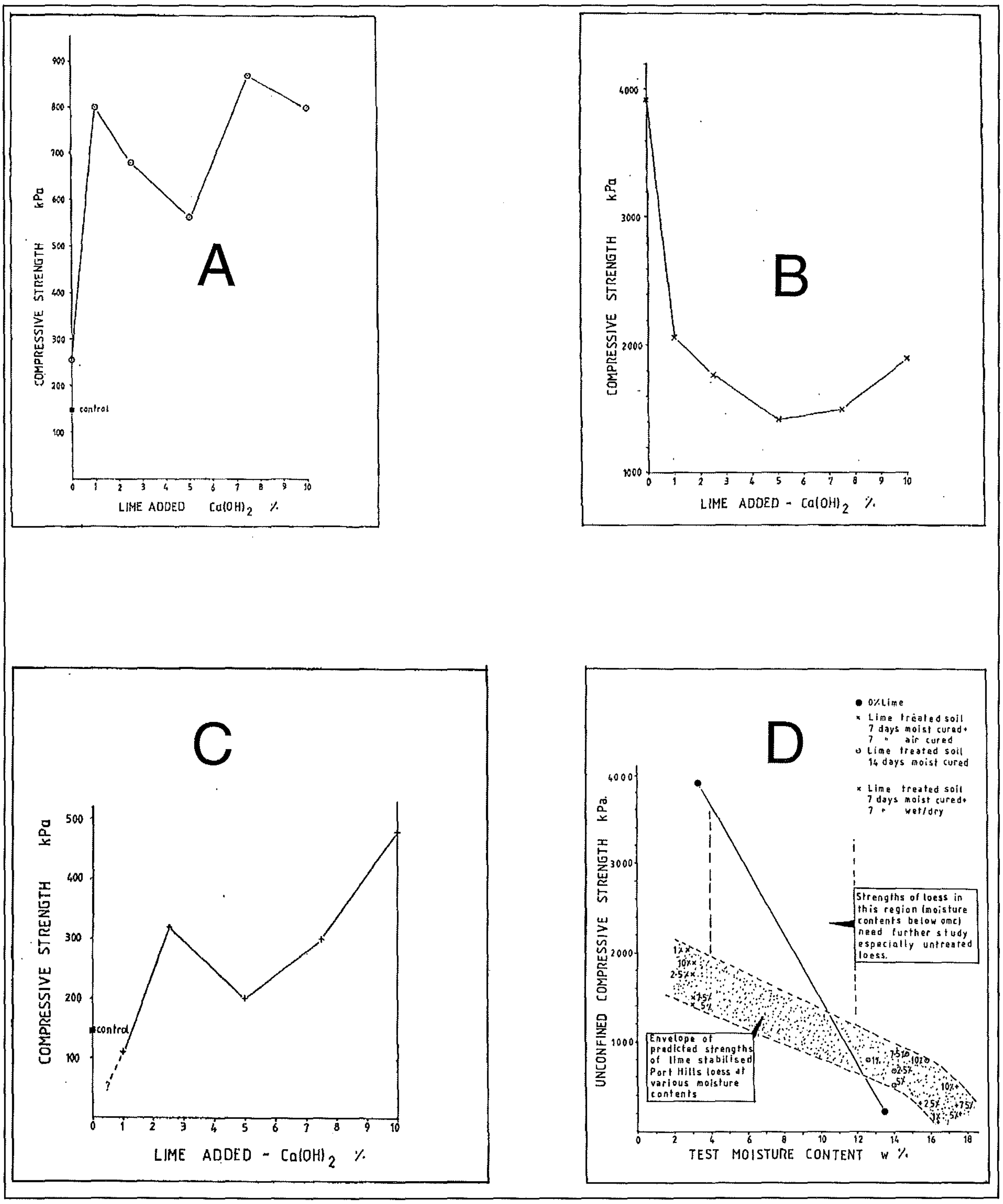


Figure 5.3 Unconfined compressive strength testing results for Westmorland stabilised loess (From Glassey, 1986): A) 14 days moist cured; B) 7 days moist cured and 7 days air dried; C) 7 days moist cured then 24 hour cycles of wetting and drying; D) water content for all samples treated.

5.2.3 Tehrani (1988)

The last and most comprehensive lime stabilisation study is provided by Tehrani (1988). Here, in-situ loess from Whaka Terrace has been tested for treated strength gains using a variety of chemical stabilisers. It is the most comprehensive of all three studies reviewed because he has sought to find shear strength parameters in contrast with the unconfined compressive strength test results, which is the conventional approach. Comparisons are also easy to make with this project as the soil is from approximately the same location. Tehrani uses both unconfined compressive strength and direct shear-box testing to fulfil his thesis aims.

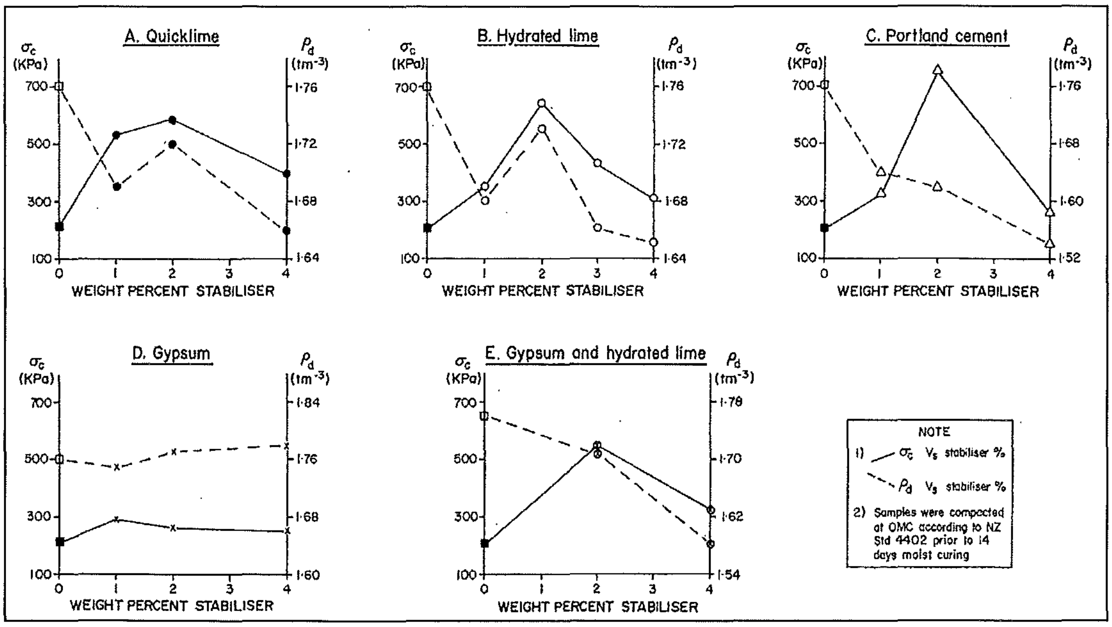


Figure 5.4 Relationships between percentage of stabiliser used, dry density and unconfined compressive strength (From Tehrani, 1988)

Bulk samples were collected from the then-proposed Whaka Terrace subdivision, recompacted at optimum moisture content, and moist cured for 14 days at 20°c. Tehrani (1988) chose five different types of chemical stabilisers to treat the Whaka Terrace soil: 1) quicklime; 2) hydrated lime; 3) Portland cement; 4) gypsum; 5) gypsum plus hydrated lime. The unconfined compression testing results, were calculated from an average of 5 for untreated samples and 2-3 for treated samples.

Samples were loaded at 0.5mm/minute. Results for the unconfined compressive strength testing and compaction tests are given in figure 5.4.

Tehrani's principal conclusions for unconfined compressive strength testing are that the application of 2% Portland cement produces the highest compressive strength, that gypsum has little or no effect, and that dry density, on average, decreases with increasing additions of stabiliser.

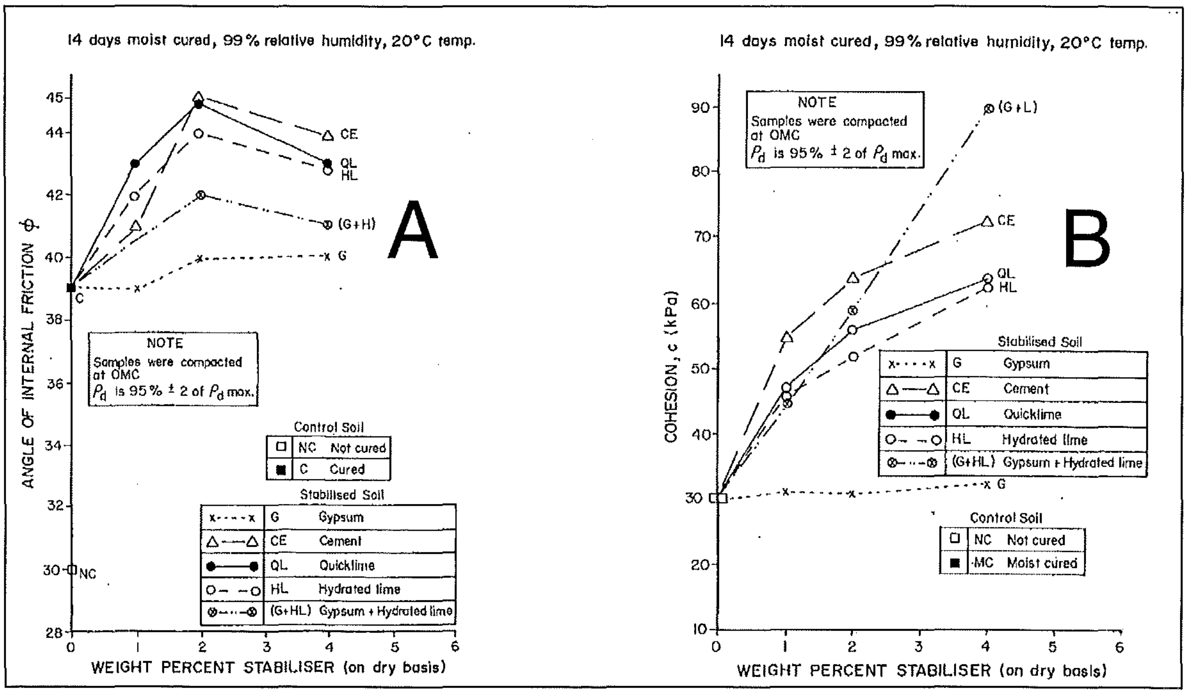


Figure 5.5 Results from direct shear-box testing of Whaka Terrace Loess; A) angle of internal friction; B) cohesion (kPa). (From Tehrani, 1988)

Recompacted loess shear box results are given in figure 5.5. Angles of internal friction show a moderate increase and peak at 2% stabiliser addition Once again Portland cement produces the best result with angles of internal friction increasing from about 39° to 45°. Cohesion for all stabilisers except gypsum doubles from about 30 kPa to about 60-70 kPa The gypsum and lime mixture records the most improvement. However, Tehrani fails to record the water content of the tested samples, although it would be a reasonable estimate that the water content would be around optimum moisture content (~14%). Tehrani states that his results agree with Glassey's (1986) data, with maximum strength being attained at around 2% addition of stabiliser, and that there is a trend of a decrease in strength after this 2% threshold. This does not quite hold true for the shear strength parameter, cohesion, which seems to increase as stabiliser is increased. Angle of internal friction, however does drop off once the addition of 2% stabiliser is passed.

5.2.4 Evaluation of findings.

Results for all three pieces of literature are comparable in trends; with each having its own specialised area of study. It was decided, from their results, that the application of 2% stabiliser would be the best choice for this project; however, the type of stabiliser is slightly more problematic. Although cement provided the best strengths of all stabilisers, as highlighted by Tehrani's (1988) thesis, it was decided to use hydrated lime as it was thought that it would be safer to work with. Hydrated lime would also produce a somewhat conservative approximation for cement, if greater strengths were needed for practical application. Also, cement once cured in the loess after 2 months would probably be very difficult to sample. It was also evident in the literature that no triaxial shear testing was done, that all treated soils had been cured under laboratory conditions not field conditions, and that curing time effects have only been examined briefly.

5.3 Whaka Terrace loess-stabilisation field experiment.

5.3.1 Site description

The site chosen was situated on Christchurch City Council land next to Whaka Terrace off Centaurus Road, on the lower Port Hills (figure 5.6). Stabilisation works had previously been installed further along the slope in the form of gabion wall baskets, and the site, historically, had the potential to fail but for all practical purposes was quite safe for field testing and posed no immediate threat to properties below. The site itself comprises a loess fill bank 10 metres wide and 25-30 metres long; with an average slope angle of 35°. The land is grassed and the

Council has attempted to plant trees for stability and shelter. West of the slope is an existing double garage, which exhibits some cracking in the concrete blocks of the rear corner backing on to the slope, indicating past instability. Beside the garage is a relatively flat piece of ground, which provided a good site for the lime stabilised test-pit.

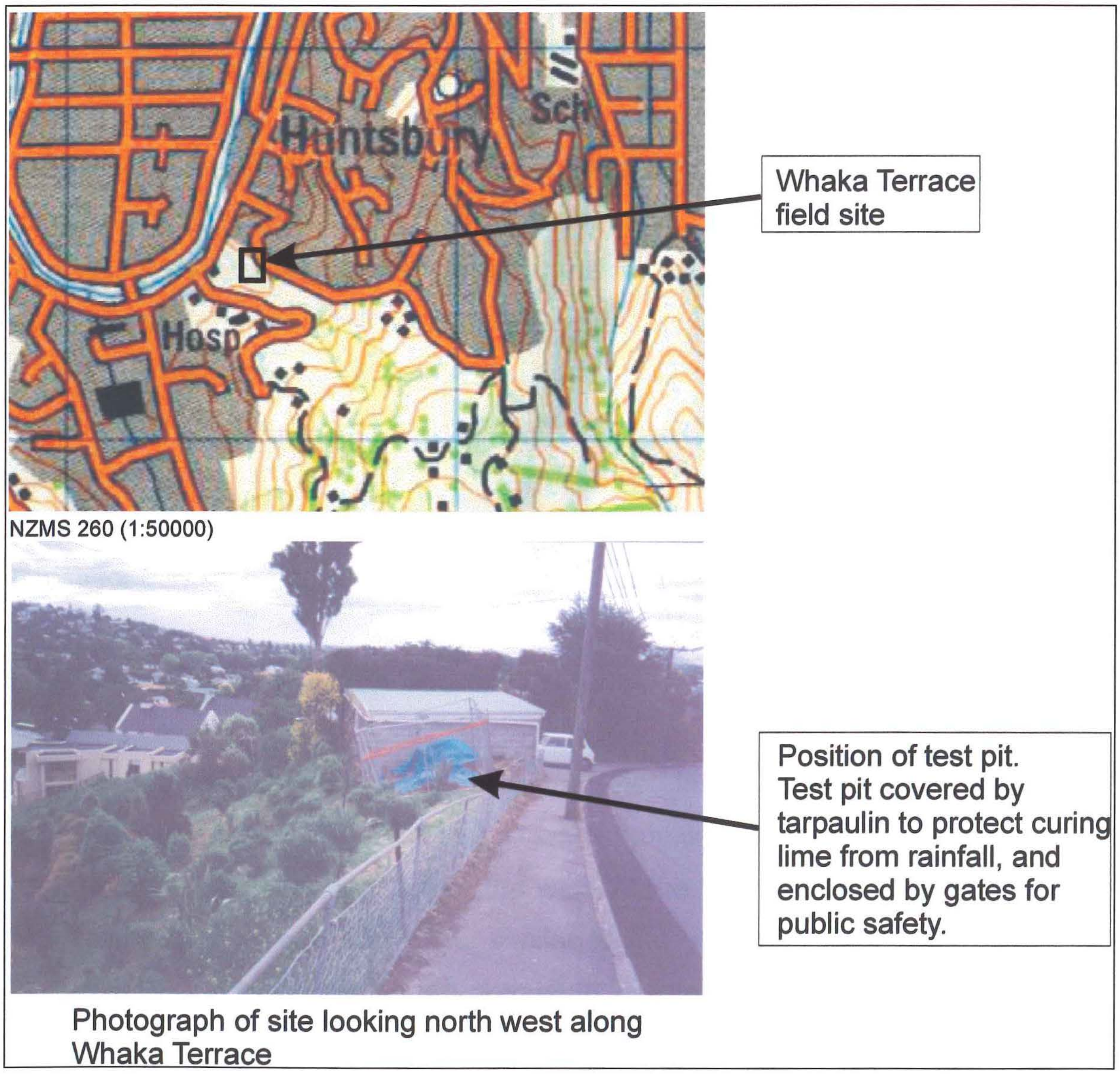


Figure 5.6 Location of Whaka Terrace field site and test pit.

5.3.2 Project aims

The project aim (stated from Chapter 1) is to determine triaxial test strength gains in uncompacted loess fill cured in the field (in situ) with hydrated lime by:

1. Excavating a test pit and applying hydrated lime to the excavated material, then recompacting excavated material back into the test pit so that lime-curing can occur under "field conditions" or "in situ curing".
2. Obtaining total stress shear strength parameters cohesion and angle of internal friction of untreated and treated loess fill at two different water contents using the triaxial test method.
3. Ascertaining the strength gain over time of treated loess in terms of cohesion and angle of internal friction.

The tests were conducted at two different water contents, 15% and "as wet as possible". So that shear strength of loess stabilised fills could be represented by "real world" parameters as well as the more improbable worst case scenario i.e. loess on the Port Hills is often not saturated to "as wet as possible". It will be seen below that cohesion and angle of internal friction parameters measured at saturation for the untreated loess are so low that if they represented what was happening within the soil of the Whaka Terrace slope the slope would not exist. Therefore results for the "as wet as possible" samples represent conservative values for cohesion and angle of internal friction in the total stress state.

5.3.3 Whaka Terrace Test Pit Description

A test pit (figure 5.9b) was excavated by backhoe in late November 2001 with the aid of BD Contractors LTD. The test pit was approximately 2m long × 1m wide × 1.5m deep. On closer inspection of the walls of the lime stabilised compaction pit there were large vein filled cracks, similar to the gammate veining found at the Duvauchelle site presented in the preceding chapter. The concordant orientation of these veins to the slope suggests that the cracks may well have been formed under tension as the newly deposited loess fill sought to stabilise itself. Whaka Terrace Loess soil described using USCS is a sandy silt, symbol ML. Soil properties are summarised in table 5.1 below. Twelve tube samples were taken vertically from the base of the test pit to test for untreated loess fill shear strength.

Field sites	Grainsize Distribution			Atterberg Limits			Physical properties		
	Sand	Silt	Clay	PL	LL	PI	γd	n	e
Whaka Terrace untreated	15.51	72.15	12.34	22.73	16.27	6.52	1.61	0.40	0.65

Table 5.1 Soil properties for Whaka Terrace Loess Fill

5.3.4 Determination of optimum moisture content for compaction

Treated loess fill was returned to the test pit and compacted for optimum strength gain. For maximum density, optimum moisture content had to be found for the treated loess fill.

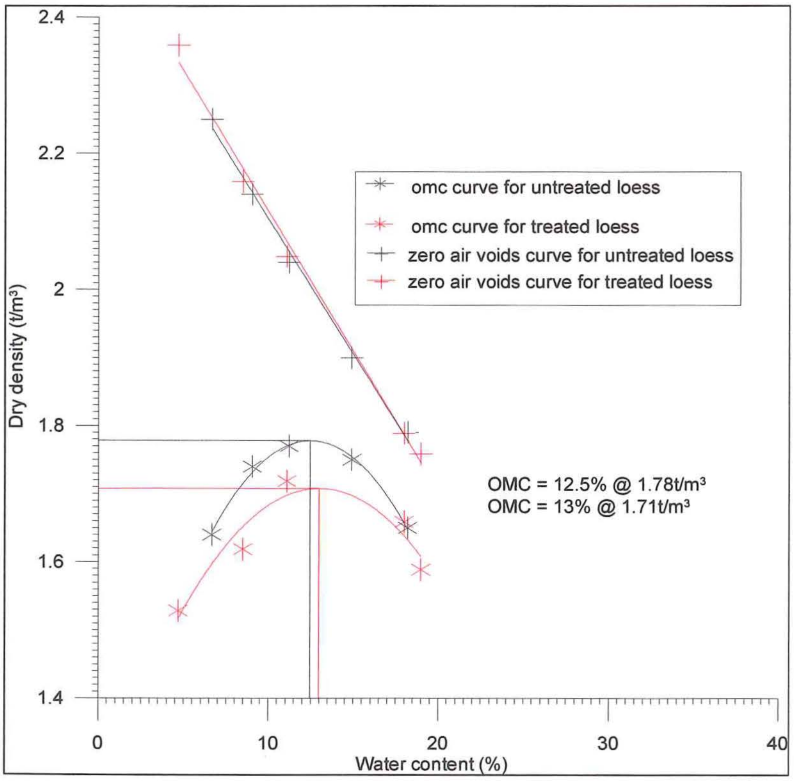


Figure 5.7 Determination of optimum moisture content for untreated and treated loess

Untreated compacted loess fill was found to have a dry density of 1.78t/m^3 and treated fill (2% hydrated lime application) 1.71 t/m^3. Optimum moisture content for the treated fill was 13.0% (Figure 5.7). In contrast dry densities measured for the untreated uncompacted triaxial test specimens were approximately 1.6 t/m^3. After completion of the dry density test for the treated samples, samples were placed in the fog room to observe the effect of fine spray and high humidity would have on the triaxial test samples. The result being no degradation, in fact, samples appeared to show no loss in shear strength at all. It was hoped that this would lead to a better wet up method than that used for all other sites as it was envisioned that treated soil would be harder to wet up using the immersion technique.

5.3.5 Whaka Terrace Test Pit preparation

With compaction parameters known it was now possible to prepare the treated soil for in situ curing in the field (Whaka Terrace Test Pit). As discussed above the amount of hydrated lime added to the loess for increased shear strength was decided at 2% by weight.

Figure 5.8 Compaction in progress. Field technician expert: Matt Smith.

An approximate calculation had to be made for the addition of 2% hydrated lime in the field. A quarter of a shovel load of hydrated lime was added to every ten shovel loads of loess fill to add the required amount of hydrated lime for 2% weight content. This is because in practical application, the worker whose responsibility it is to prepare the soil would probably not have enough time to weigh out accurately each addition of hydrated lime to a corresponding amount of loess for a compacted layer. A simple and effective method had to be found and used to simulate “field practical application”. With this inaccurate field method in mind, strength gains for field treated loess were still found to be substantial.

Once these preparation parameters were known the previously excavated fill was placed back into the lime compacted pit for trial (figure 5.9a). The first metre of compacted loess was untreated as it was felt that that amount of soil would not be needed for sampling. Four 10-15 centimetre intervals were then marked off on the side of the pit wall so that four layers could be compacted separately to comprise the sampling zone using a Whacker Compactor (figure 5.8). A sample was taken for water content, which was found to be 13.6%, just wet of optimum moisture content.

The pit was filled to just 20cm below the top and covered over with a plastic tarpaulin. Tube sampling took place in the bottom three layers and a total of 32 samples were extracted from the treated loess at intervals of 1 week, 1 month and 2 months approximately depending on weather conditions at the time of sampling (no samples were collected in the rain, which was often at this time). No field wet up process involving “soak pits” were needed to collect intact triaxial test samples. It is not known why this is but the loess acted with greater ductility when penetrated by the sample tubes. All samples collected except two were tested for shear strength.

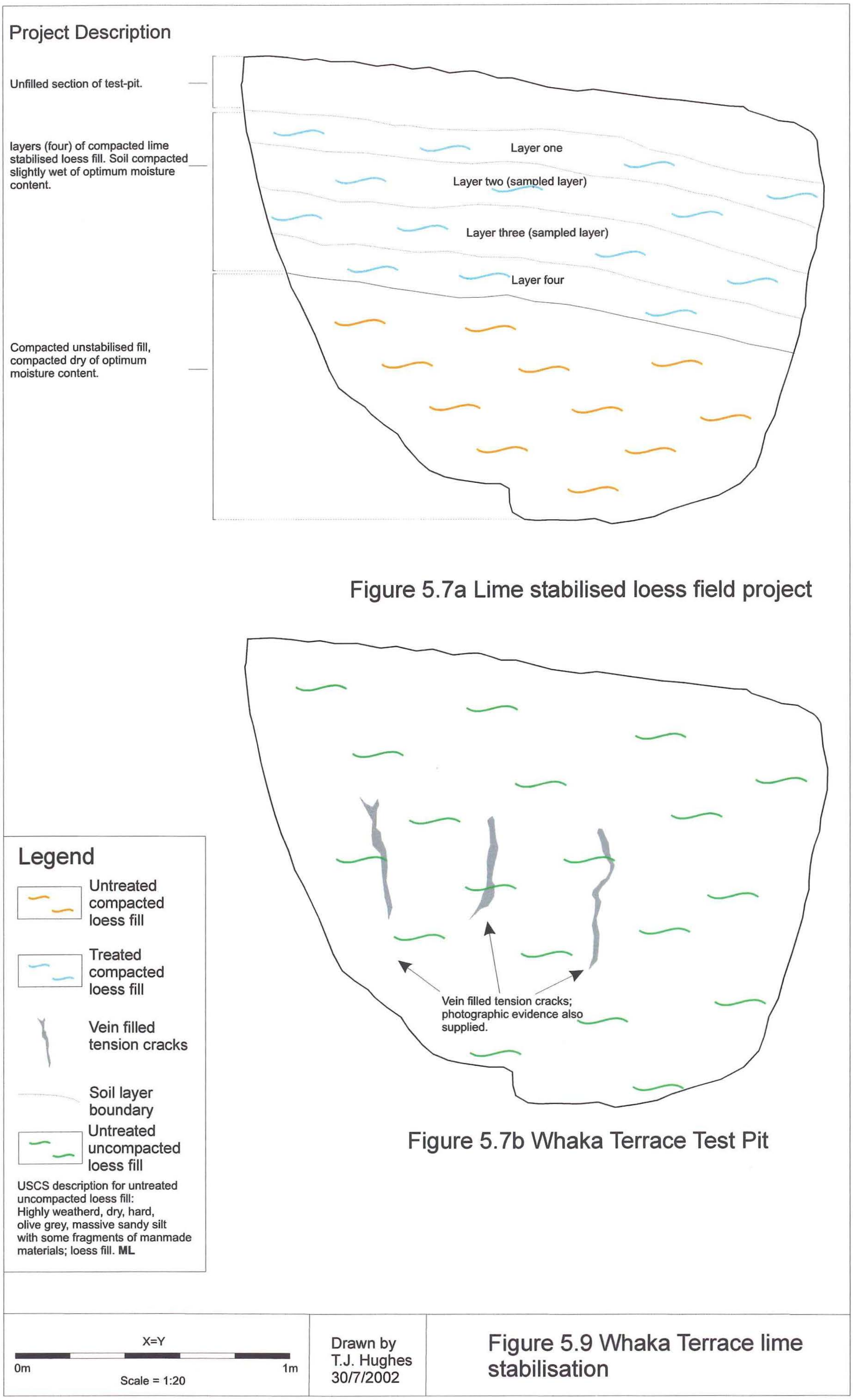


Figure 5.7a Lime stabilised loess field project

Figure 5.7b Whaka Terrace Test Pit

Figure 5.9 Whaka Terrace lime stabilisation

5.3.6 Triaxial test results and analysis.

Shear strength parameters were determined using the standard undrained unconsolidated triaxial test, without measurement of pore pressure. T-S plots were graphed using the results from these tests and are presented in figure 5.10 Raw data are presented in Appendices 1, 2 and 3. Angle of internal friction and cohesion were calculated in the method outlined in Chapter 4. A summary of shear strength parameters, angle of internal friction and cohesion are presented in Table 5.2 below:

	Water content	cohesion (kPa)	Angle of internal friction
Untreated loess	15.40% 20.70%	0.0 7.0	30.6 11.5
7 day lime cured loess	16.90% 19.10%	30.7 4.0	30.7 23.4
25 day lime cured loess	16.10% 17.30%	63.2 16.5	25.1 27.2
68 day lime cured loess	10.20% 19.20%	92.1 24.8	42.7 30.3

Table 5.2 Shear strength parameters for Whaka Terrace treated Loess at increasing time intervals

Tests were conducted at two different water contents so that a shear strength dependence on water content could be established (if any). As can be seen from Table 5.2 all time intervals show decreasing angle of internal friction and cohesion with increasing water content. Actual water contents for the 15% nominated water content were not targeted well, and in the case of samples collected after 68 days water contents under shot the nominated water content by 5% However shear strength difference between the two water contents show a steep trend of increasing water content to decreasing cohesion and angle of internal friction.

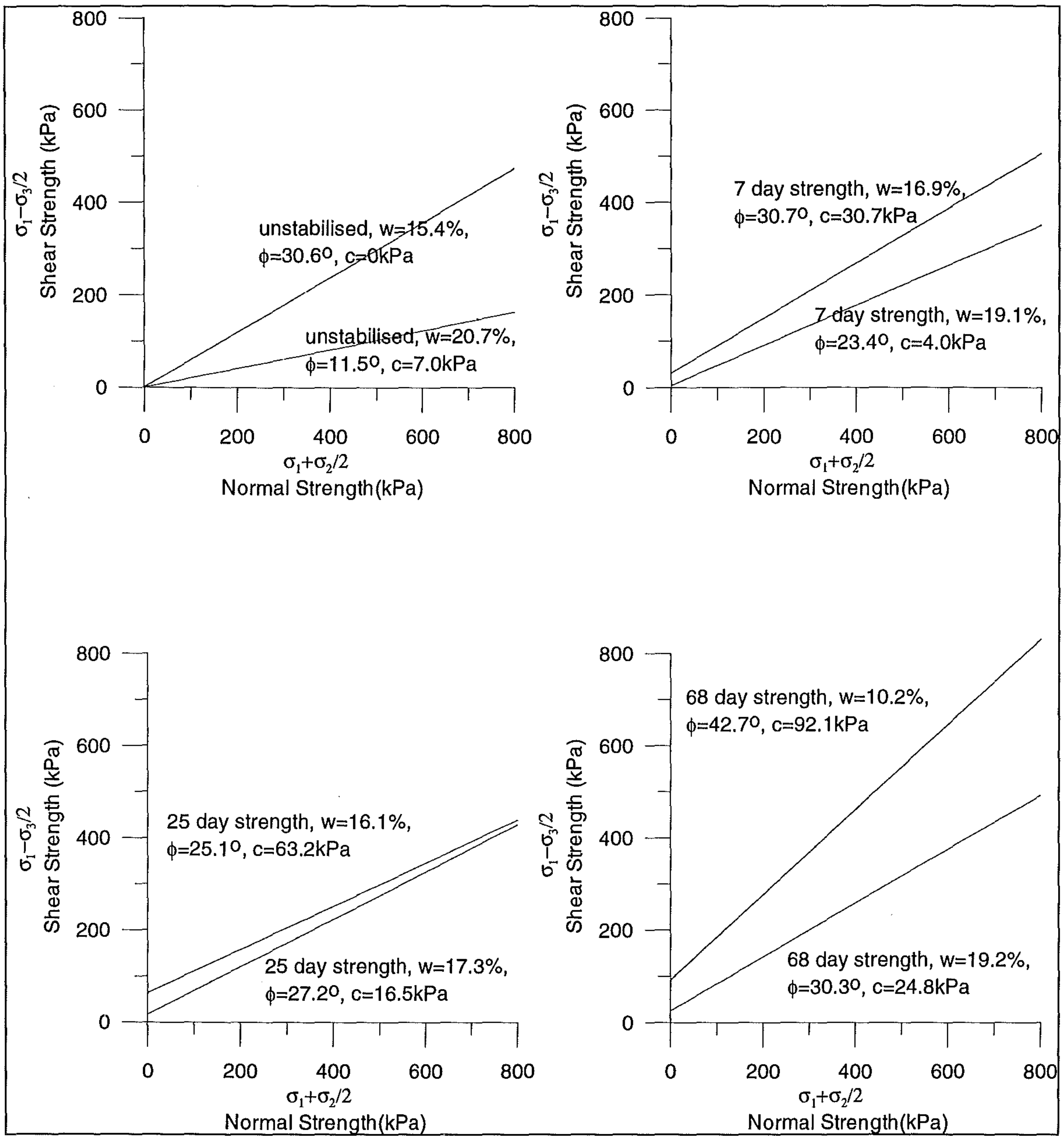


Figure 5.10 T-S plots for hydrated lime-treated Whaka Terrace loess. Separate graphs shows shear strength relations for different time intervals.

Figure 5.11 shows angle of internal friction and shear stress graphed against time. Trends show that both shear strength parameters follow an increase with time, although there is a slight reversal of trend for cohesion at "as wet as possible" and angle of internal friction for a nominated water content of 15%. For the other two trends there is a rapid rate of increase for the first week and then the rate of increase tends to level off.

Increases of strength are up to three times the amount for untreated loess, and have angles of internal friction and cohesion similar to that found for the primary airfall in situ loess test results presented in Chapter 4. It can be concluded that the bulk of strength gains occur in the first week or so of application, but that strength gains are ongoing, although it can't be concluded from this data set when strength gains stop.

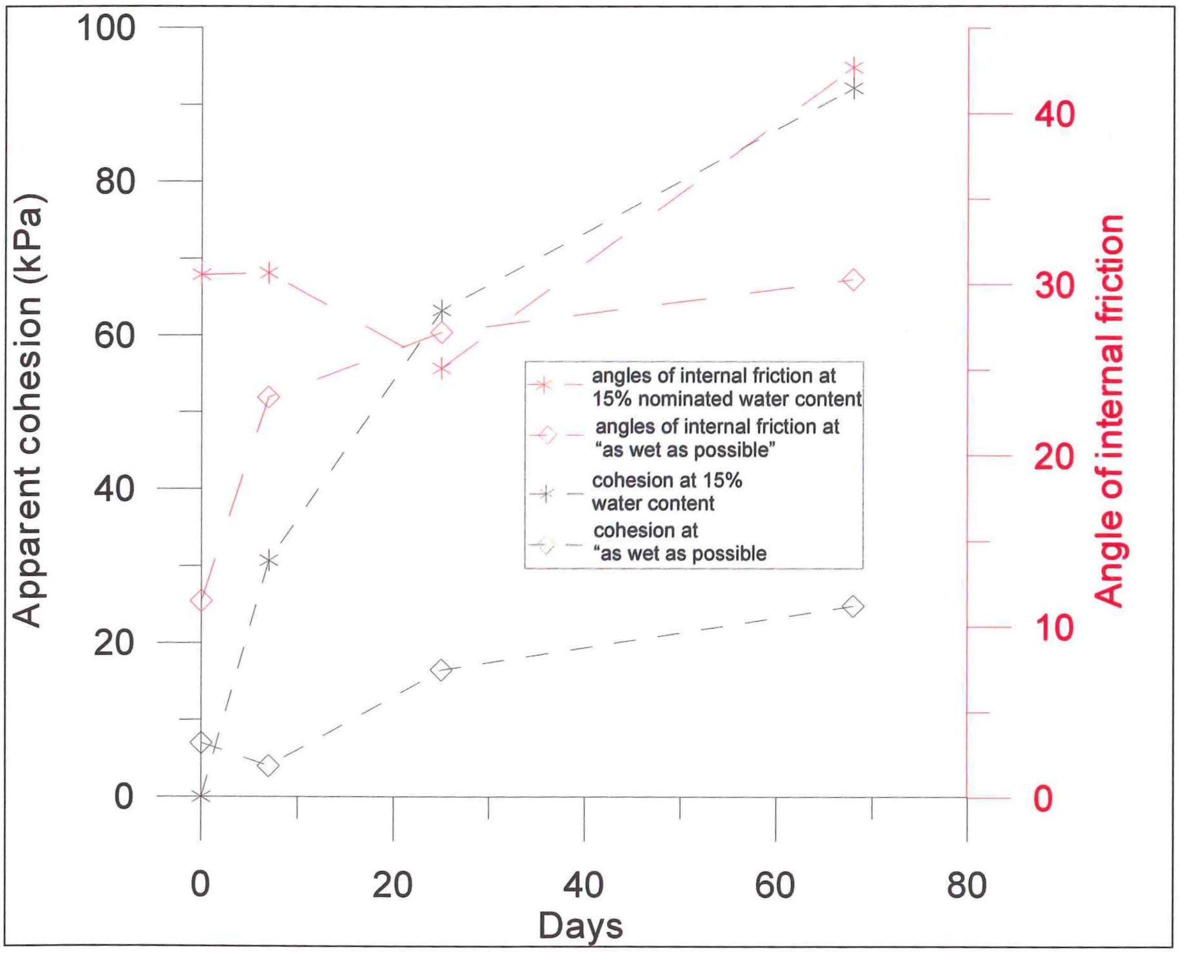


Figure 5.11 Summary of strength gains for Whaka Terrace loess over a 68 day period.

5.4 Synthesis.

1. From previous literature of lime stabilising effects on shear strength of Port Hills loess, it was decided that an addition of two percent hydrated lime would be appropriate to trial a lime stabilised shear key on a loess fill slope.

2. Optimum moisture content for Whaka Terrace lime stabilised loess was calculated to be 13.0% with a resultant density of $1.71 t/m^3$.

3. Triaxial tests conducted at different water contents showed a shear strength dependency on water content. Trends showed that as water content increased angle of internal friction and cohesion decreased.

4. Triaxial testing of hydrated-lime stabilised loess revealed an increase of shear strength parameters for loess cured in the field for 68 days. Loess fill cohesion and angle of internal friction increased by a factor of three.

CHAPTER 6

Summary and Conclusions

6.1 Shear strength dependency on water content

The primary aim of this thesis was to determine evaluate strength dependency on water content for Banks Peninsula soils. This was achieved by selecting sites on the Peninsula, extracting samples from these sites, testing them at different water contents in the laboratory, and analysing the laboratory results for the shear strength parameters cohesion and angle of internal friction in the total stress state.

The sites that were selected for testing were: 1) Moncks Spur; 2) Stonehaven Subdivision; 3) Worsleys Spur; 4) Whaka Terrace; and 5) Duvauchelle. The first four sites listed were collectively termed Port Hills sites, but can also be classified using Griffiths (1973) type sections as Birdlings Flat Loess whilst the fifth is sited in distinctly different Barry's Bay Loess.

Samples from all sites were extracted from the "P-layer" (Hughes, 1970) in test pits, machine-dug especially for sampling. Samples were collected by pushing 35mm diameter stainless steel tubes vertically into the "P-layer", and a field "wet-up" technique was developed as the loess was too brittle and dry to sample. This involved hand-digging small "soak pits" in the base of the test pit, filling them with water, and letting the water slowly infiltrate the underlying soil. The loess became softer (but did not deform) and easier to sample to produce intact specimens for the laboratory.

Different water contents in the loess samples were achieved by immersing the stainless steel tubes containing the sampled loess in water to get the samples "as wet as possible", extruding the samples from the stainless steel tubes and drying them to nominated water contents, which were 6, 10, 14% and "as wet as possible". Samples were then wrapped in plastic wrap and tin foil, placed into an airtight container, and left for one week before strength testing to allow the water in the sample to disperse evenly throughout.

Shear strength was tested in the laboratory using the unconsolidated undrained triaxial test method without measurement of pore pressure. The triaxial test was chosen because it best modelled undrained high rainfall failure, and samples were easy to prepare for water content variation. Three cell pressures of 50, 100 and 150 kPa were used to determine a Mohr-Coulomb failure envelope (and subsequently cohesion and angle of internal friction) for all samples. These cell pressures correspond to depths of 3, 6 and 9m, which covers the range of thicknesses seen in Banks Peninsula "P-layer" Loess. Also, three cell pressures were chosen instead of more to improve the repeatability of testing results.

Mohr-Coulomb failure envelopes were calculated using an unconventional method of relating shear stress ($\sigma_1 - \sigma_3/2$) to the Mohr-Coulomb envelope by a simple geometric relationship. This method was used because fitting a Mohr Coulomb envelope to Mohr circles for all tests completed at a nominated water content was too subjective. Actual water contents measured for test completed in a nominated water content group (measured after the test was completed) were averaged and this figure was assigned to the angle of internal friction and cohesion which were determined from their respective tests.

Principal conclusions obtained from laboratory strength testing of Banks Peninsula loess are:

- A new trend of increasing cohesion and decreasing angle of internal friction for increasing water contents in the total stress state was found for all Port Hills sites. A maximum for cohesion was reached at approximately 12-14% water content after which cohesion dropped; angle of internal friction continued to fall. Maximum cohesions recorded for primary airfall in situ loess were approximately 50kPa.

- This new trend was not found for Duvauchelle Loess and was therefore interpreted to be distinct from Port Hills Loess. Duvauchelle Loess cohesion was found to decrease exponentially with increasing water content and angle of internal friction decreases linearly with increasing water content, reaching zero at about 12% water content. However, a new method was used for c and ϕ calculation based on Kane's (1968) work and the trend of cohesions increasing at lower contents to a maximum was now seen for Duvauchelle Loess. This maximum was recorded at 209 kPa after which cohesion decreased exponentially, but did not reach zero and had a cohesion of 48 kPa at 16% water content. It was maintained, after Kane's method of calculation, that Duvauchelle Loess was still distinct from Port Hills because it had a far higher cohesion and far higher clay content. Trends for Duvauchelle Loess showed similarity to Kane's Iowa Loess and clay contents were also of a similar value, 19.5% and 19% respectively.

- Primary airfall in situ loess of the Port Hills sites (Moncks Spur and Worsleys Spur) is distinct from the loess fill and colluvium (Whaka Terrace and Stonehaven Subdivision) because primary airfall in situ loess displays far higher cohesion. Maximum cohesions are 11 kPa for both fill and colluvium.

6.2 Lime Stabilisation at Whaka Terrace

The secondary aim of this thesis was determine strength increases over time and at different water contents for hydrated lime-treated field cured loess fill at Whaka Terrace. This was achieved by:

1. Excavating a test pit and taking untreated samples for strength testing so that they could be compared to treated samples
2. Adding 2% hydrated lime by weight to the excavated material, mixing and recompacting (using a Wacker compactor) back into the test pit at optimum moisture content.
3. Extracting test samples at time intervals of 1 week, 1 month and two months.
4. Conducting triaxial shear testing on all of the collected samples.

5. Analysing laboratory data for trends in shear strength dependence on water and determining the shear strength gain over time using the calculation method used for the primary aim.

Principal conclusions from laboratory testing of lime treated loess fill are:

- A decrease in cohesion and angle of internal friction in the total stress state with increasing water content was seen for all samples tested at different time intervals, except for untreated loess, which followed the general Port Hills Loess shear strength trend as stated in the section above. The difference for strength of treated loess fill between 15% water content and "as wet as possible" was approximately 30 – 40 kPa for cohesion and 5° for angle of internal friction. The highest recorded cohesion for treated loess fill was 92 kPa at 10.2% water content, which was far higher than any cohesions recorded for primary airfall in situ loess.

- An increase of shear strength parameters cohesion and angle of internal friction for stabilised loess with time. This trend shows a rapid rate of increase in the first week of lime application and then slows down. Both angle of internal friction and cohesion for treated (field cured) loess showed strength increases of up to 3 times more than the untreated loess after a period of 68 days. Cohesions increased from 7 kPa to 25 kPa and angle of internal frictions increased from 11° to 30°.

6.3 Recommendations for Further Research

Two major themes have been highlighted in this thesis, one has been researched extensively and the other has not been mentioned but would most probably be foremost in the readers mind. The first, in terms of slope stability is at exactly what water content does loess show a complete loess in shear strength. It is hoped that this thesis has managed to provide data and analysis that will help to solve this problem. The second is exactly how much water is contained within a loess deposit, prior to and upon failure; does it need to be saturated? Some ideas on how to research these themes are given below:

1) Obtaining effective stress parameters at different water contents using methods outlined by Fredlund et al (1978). This has never been done before in a quick undrained shear test, however, special equipment would have to be imported to do this. Determining effective stress parameters using this method would also evaluate soil suctions in the loess and a critical water content (developed by Kane, 1968) could be established. This method would also further investigate the new trend seen in Banks Peninsula Loess, which is presented in this thesis.
2) Investigating shear strength parameters c and ϕ at water contents lower to that tested in this project to determine whether trends seen for Port Hills Loess trends in this project are an artefact of the analysis methods used or something else.
3) Investigating the brittle-ductile change, this is thought to occur at between 10-15%. This could also be compared to Kane's (1968) critical water content to see whether any relationship exists between the two.
4) A more detailed assessment of loess shear strength parameters for Barry's Bay Loess, which has reported to have distinctly different strength behaviour to that seen in Birdlings Flat Loess. Also instability problems in terms of strength are more prevalent in Barry's Bay Loess as the rainfall in inner harbour areas (where this deposit occurs) tends to be greater than the surrounding flanks of the Banks Peninsula Calderas.
5) Investigation of soil moisture fluctuations in Banks Peninsula Loess with special emphasis placed on wetting fronts in times of large rainfall. If it is assumed that shear strength of Banks Peninsula loess is solely dependent on soil water contents, then understanding how water regimes behave within the soil will best determine how stable any Banks Peninsula loess deposit is.
6) Investigations of loess failures by observation in the field and "real-world" modelling, either in the field or laboratory. This would ultimately evaluate loess shear strength behaviour via back calculation of shear strength parameters and observing water behaviours within a failing loess deposit.

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